Controlled Piledriving Above and Under Water With A Hydraulic Hammer

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ABSTRACT

Investigations of phenomena that occur during piledriving have led to the realization of a hydraulic hammer, the HYDROBLOK®, and to a new wave theory-based computer program, both different in a number of aspects from those commonly in practice today.

The characteristic shape of the impact diagram of this hammer leads to better driving results in a fully controlled fashion and prevents damage.

The hydraulic system makes possible underwater utilization remotely controlled; also with the large 1 million ft.lb. Hydroblok.

Specific Hydroblok characteristics are taken into full account in the new HDB-computer program that simulates in a realistic manner all movements and stresses as they develop in the hammer-pile-soil system during driving.

Values of dynamic soil resistances and their distribution can be learned from the driving itself.

Advance driveability studies prior to driving become more reliable.

This paper gives information on three specific subjects. Firstly some technical aspects on Hydroblok hammers and their submarine use are shown.

Secondly some theoretical and applicational aspects of the HDB-piledriving computer program and finally how in-the-field measurements and loadtest results closely relate to the values that the program was able to calculate with data that could be derived from the driving itself.

INTRODUCTION

Piledriving causes force waves to travel. Their magnitudes generally are only being known roughly since they depend on many factors, such as hammer impact and soilresistances.

In the attempt to avoid concrete piles being damaged by tension-waves it became necessary to deliberately influence the shape of the impact diagram; this gave birth to a hydraulically driven double-acting hammer of unconventional design, the HYDROBLOK. Among its specific characteristics it is perhaps the capability to drive piles underwater with exactly the same high efficiency as above water that makes application offshore so attractive.

References and illustrations at end of paper.
Concurrently a piledriving theory was required that should process to its fullest extent all Hydroblok properties as well as their effects on the pile. Fully operational now is the HBM-computer program that really gives credit during driving to the interactive behaviour of hammer, anvil, pile, possible followers and soil. Many tests and measurements during piledriving jobs have rendered indispensable information enabling theory, computations and predictions repeatedly to be compared with reality.

Thanks to this mathematical model it became possible to design a piledriving hammer with the most desirable characteristics.

**HAMMER DESIGN PHILOSOPHY**

Since the first materialised Hydroblok prototype was brought into operation in 1969 quite some innovations were realised. Cross-section fig.1 shows the main components of a modern Hydroblok hammer. Principal idea behind the concept is the built-in prestressed Nitrogen filled buffer that enables the "dropweight" to continue its downward movement immediately after the moment of impact until all energy generated from the downward velocity has been fully released to the pile. During all this time - 10 to 20 milliseconds - the nearly constant bufferforce/impactforce remains active on the pile. The highly desirable Force-Time or impact-diagram has so been given its shape; it is realistically shown in fig.2 and diagrammatically in fig.3.

The wavetheory learns that a pile will only penetrate as long as total soilresistance remains below twice the downward force applied to the pile. Apart from the peakforce (fig.2) this downward force for the Hydroblok is represented by the bufferforce, its value being known at all times to the operator, enabling him to remotely accommodate his bufferforce to changing soilresistances during driving in order to maintain the maximum possible part of the impact-diagram available for effective penetration.

Hydroblok hammers can withstand high blow-counts without damage; the built-in buffer not only provides means for perfect controlled piledriving but it also protects the hammer from being damaged by uncontrolled peakforces.

Hydroblok energies are stated as pure net values since a device could be developed that measures velocity V at moment of impact; where mass m of the "dropweight" is known energy-per-blow can now simply be calculated with the formula ½*m*V², fully doing away with the problem of guessing at hammer efficiencies. This specifically applies to the Hydroblok hammer because there is only steel to steel contact beyond the spot of impact and for steel piles there is no cap filling material with undefined properties.

**HYDROBLOK OPERATIONAL DETAILS**

Double-acting hydraulic cylinders inside hammer casing prime the "dropweight" down with an acceleration of 1.6 times gravity, the weight of the hammer casing acting as counterweight. Inevitable constructional weight because of the demand for a rigid and completely closed housing is so given extra function. This explains Hydroblok's favourable energy-per-blow/selfweight ratio (table 1).

The hammer is powered by means of high pressure fluid transported through a hydraulic hose. After having transferred the energy to the double-acting cylinders the fluid is returned through a second hose to the powerhouse.

This closed-circuit energy transportation system is suitable to bridge greater distances, through air as well as in the water also into greater depths.

A third hydraulic hose, the buffer hose, enables the operator to know explicitly the aforementioned impactforce that he can vary at liberty without stopping the driving. The hammer being invisible and inaudible underwater unveils constantly its correct bufferforce.

Moreover the operator knows exactly from his controls when to accommodate bufferforce to (unknown) changing soilresistances. For shortness sake it must be assumed in this contribution that the Hydroblok-hammer principles are known, such as how exactly the interrelations are between energy-per-blow (ft.lb.), impactforce (lbs or kips) and the impact-duration (milliseconds) and how these can be widely varied during driving. In earlier publications these specific Hydroblok properties have been described in detail (1,2,3).

In order to secure the same hammer performance when working underwater it is necessary that the hammer casing in which the "dropweight" moves up and down is permanently kept free from water, as well as the spot where the impact is delivered to the anvil. A number four hose therefore is required to transport air or inert gas into the hammer casing; fig.4 shows that the hammer casing extended with the pile sleeve being open at its lower end to receive the top of the pile is at any given waterdepth subjected to the same differential pressure equivalent to the hammer height "a" ft. water-column.

A simple automatic system controls fully automatically the compressed air supply to the hammer casing during submergence.
It is suitable for great waterdepths (fig. 5). The pile sleeve also functions as perfect guidance for the pile. The hammer may sit freely on top of the pile provided the pile has sufficient lateral stability. From the driving itself there are no force components that act sideways on the pile. Besides, the relative axial movements between the hammer and the pile are practically negligible during driving; this sounds rather unbelievable but it is true for a Hydroblok (4). To maintain stability under water this double-acting hammer requires ballast to compensate for the extra buoyancy that results from the air inside the hammer casing.

In most offshore applications specific guideframes are required to compensate for relative movements. As a rule these guideframes demand considerable constructional weights because of stiffness requirements of the hammer-guideframe combination. The cylindrical Hydroblok casing has great stiffness of its own and the heavy upper and lower flange provide perfect means for a simple and relatively light type of guidewhich frame is of sufficient strength to satisfactorily fulfills the same compensating requirements. Fig. 6 shows the HSM-3000 type hammer with its standard guidewhich frame. Weights are given for underwater application as well as for use above water. It has to be borne in mind that HSM-3000 is a one-million ft.1b. (rated) energy pile-driving piece of equipment. Table 2 finally gives some main data of the 500, 1500 and 3000 types.

PILEDRIVING THEORY AND PHILOSOPHY

As said before the development of this controlled system of pile driving demanded a pile driving theory with the capability to describe exactly what really happens when stress waves propagate downward and upward in the pile as well as in the hammer. Without the aid of such a computer program today's Hydroblok hammers would not have been possible.

Furthermore it was judged to be essentially important to incorporate only as few soil-parameters as possible. Their values were derived from many pile driving tests. Driving a pile into soil initiates stress waves to run in the pile and in the hammer resulting in movement of the pile and in modification of the movement of the hammer. The pile movement causes the soil to develop resistances (skinfriction and point-resistance) against deformation with the result that equally in the soil stresswaves are propagated being of a complicated nature. Fortunately the pile driving theory does not require these stresswaves in the soil to be taken into consideration explicitly.

Resistances to penetration of the pile may be taken as boundary values.

The pile driving theory therefore is based on

- a - the theory of propagation of stress waves in the hammer, the anvil and the pile,
- b - a conception of the laws that govern the development of soil resistances relative to movements of the pile, as these could be derived from pile driving tests.

SOILPARAMETERS

Pointresistance in sands does develop within a few milliseconds from a certain initial value to a limitvalue remaining fairly constant as long as the pile tip moves (fig. 7). This limitvalue depends on the nature of the sand, its state of stress and in general the geological history of the location. This limitvalue in sands has been found to be fairly equal to the mean value of the cone resistance that one finds with the Dutch Cone Penetrometer. The mean value has to be taken in the zones where distinct shearplanes occur. This depends on the area where the pointresistance is acting. Unit pointresistance variations from 30-300 kg/cm² have been found.

The path from initial value to limitvalue may be taken linear for calculation purposes.

Pointresistances in clays are fairly constant as long as the pile moves, its value generally being less than in sands, except possibly in overconsolidated clays.

To come to a good understanding of skinfriction this phenomenon must be dealt with more in detail here. The load-deflection diagram for skinfriction is generally assumed to consist of an "elastic" part and a "plastic" part as is shown in fig. 8. The "elastic" part being valid for deflections u between zero and a certain value q (the quake) the resistance W equals:

\[ W = \frac{W_1}{q} u \quad \text{for } 0 < u < q. \]

Comparison of this idealized diagram with the results of measurements, as for instance published by Baa (5) shows that:

- resistance is not zero at zero-deflection,
- for very small deflections the resistance increases proportionally to the deflection (Baa(5), fig. 2); dotted line Wm in fig. 8.

Such a diagram can be idealized. See fig. 9. In the case of pile driving, however, this diagram must be dynamically translated. For the static diagram of fig. 9 is valid:

\[ W = W_o + \frac{W_1 - W_0}{q} u \quad \text{for } 0 < u. \]
time \( t \) = \frac{1}{2} \cdot V(x', y') \cdot t = \frac{1}{2} \cdot V(x', y') \cdot \sqrt{\frac{2 \cdot g \cdot y}{V(x', y')}}

\[
W(x) = \frac{1}{2} \cdot V(x', y') \cdot t
\]

\[
W = \frac{1}{2} \cdot V(x', y') \cdot \sqrt{\frac{2 \cdot g \cdot y}{V(x', y')}}
\]

where

\[
(\frac{1}{2} \cdot V(x', y'))^2 = \frac{1}{2} \cdot V(x', y') \cdot \sqrt{\frac{2 \cdot g \cdot y}{V(x', y')}}
\]

Notes:
- This expression in practice is applied
- However, the proportion to the distance
- is modeled by 1/2 the distance
- and not according to the formula
- of the material and density of the structure.
- This leads to a constant proportion
- of the distance of each layer on a distance
- of the structure, which is the very small
- difference of material and density.
- The proportion of the distance must be that in dynamic
- components and not on the deformation
- or the structure of the structure.
- This is depicted in Fig. 10.

\[
\text{V}_{\text{total}} = \sum_{i=1}^{n} \text{V}_{\text{individual}}
\]

\[
(\frac{1}{2} \cdot V(x', y'))^2 = \frac{1}{2} \cdot V(x', y') \cdot \sqrt{\frac{2 \cdot g \cdot y}{V(x', y')}}
\]

\[
W = \frac{1}{2} \cdot V(x', y') \cdot \sqrt{\frac{2 \cdot g \cdot y}{V(x', y')}}
\]

Superscript in the fraction equals to the power of

\[
\frac{1}{2} \cdot V(x', y') \cdot \sqrt{\frac{2 \cdot g \cdot y}{V(x', y')}}
\]

Terms are also found in different

\[
V \cdot \frac{1}{2} \cdot V(x', y') \cdot \sqrt{\frac{2 \cdot g \cdot y}{V(x', y')}}
\]

The expression is the fraction of the total

\[
\frac{1}{2} \cdot V(x', y') \cdot \sqrt{\frac{2 \cdot g \cdot y}{V(x', y')}}
\]
FUNDAMENTAL WAVE THEORY

Philosophy behind this theory is that the basic wave theory can be expanded straight-forwardly to make it incorporate the real complex phenomena. First the simple basis.

A pulse applied to one end of a rod causes a stresswave to run towards the opposite end with celerity \( c = \sqrt{E/\rho} \) (E=Young's modulus and \( \rho \)=density). Now it will be reflected depends on the condition of this end; the celerity of the reflected stresswave, however, will be the same (fig.12).

The two stresswaves together result in stresses and particle velocities, both being functions of place and time. Assuming no friction nor internal damping being present waves will run undisturbed (i.e. intensity as well as intensity distribution remain unaltered).

At a distance \( x \) from the primary end at time \( t \) the wave running in the outward direction has an intensity \( \sigma^+ \), the backward wave has an intensity \( \sigma^- \), the total stress \( \sigma \) being equal to \( \sigma = \sigma^+ + \sigma^- \).

The velocity \( v \) of the rod at this place and time is equal to

\[ v = v^+ + v^- = \frac{E}{\rho} (\sigma^+ - \sigma^-). \]

Introduction of force \( F = \rho A \sigma \) (A=cross section of the rod) modifies both formulae as follows:

\[ F = F^+ + F^- \quad \text{and} \quad v = \frac{E}{\rho A} (F^+ - F^-) = \frac{E}{Z} \frac{F^+ - F^-}{Z} \]

\[ Z = \frac{E}{\rho} \] being the impedance of the rod.

In case the rod already had a velocity \( v_0 \) before the pulse was applied the formula changes to

\[ v = v_0 + \frac{F^+ - F^-}{Z} \]

With these simple formulae it is possible to analyse what happens:

- when there is an impact between two rods having impedancies \( Z_1 \) and \( Z_2 \) and velocities \( v_0^1 \) and \( v_0^2 \) (fig.13),
- when the waves reach a discontinuity of the impedance (fig.14),
- when a wave is reflected (fig.15).

Foundation piles may basically be considered such as a rod and also the hammer (with some approximations), were it not that there always is skinfriction.

INTRODUCTION OF SKINFRICITION

The following philosophy enables skinfriction to be projected in a simple engineering manner.

In reality skinfriction is distributed over the entire embedded length of the pile. For the sake of simple analysis it may be assumed to act at discrete points along the shaft on the condition that the concentrated skinfriction at a certain level is equivalent to the skinfriction distributed over a small adjacent area.

Assumed these friction-points to be distances \( x \) apart the concentrated friction must be equivalent to the friction distributed \( \frac{1}{x} \) above and \( \frac{1}{x} \) below the friction-point.

In this case the parts of the pile with lengths \( x \) between the friction-points may be considered as part of the rod where no friction exists with the consequence that all formulae mentioned so far still remain in full force.

The analysis can now be extended as follows. See fig.16. Waves arriving at a certain friction-point level are \( F^1 \) from above and \( F^2 \) from below, both assumed to be known. The friction \( W = W_o (1 + \alpha v) \), \( W_o \) and \( \alpha \) assumed to be known.

Waves moving away from this friction-point are \( F^1 \) upwards and \( F^2 \) downwards.

The following equations are valid now:

\[ F^1 + F^1' = F^2 + F^2' + W \]

\[ F^1 - F^1' = F^2 - F^2' = \frac{v}{Z} \]

\[ W = \frac{W_o}{\alpha v} \]

Solution of these equations leads to:

\[ W = \frac{W_o Z + (F^1 - F^2)}{Z + \alpha v W_o} \]

\[ F^2 = F^1 - \frac{3W}{Z} \]

\[ F^1 = F^2 + \frac{3W}{Z} \]

\[ v = \frac{F^1 - F^2 + \frac{3W}{Z}}{Z} \]

It is important to observe that each time when a friction-point is encountered
- the downward wave is reduced by \( \frac{3W}{Z} \)
- the upward wave is increased by \( \frac{3W}{Z} \).

This course applies to positive pile velocities (downward in fig.16). For negative \( v \) (upward) the sign of \( W \) is reversed.

If the pile at a certain instant is at rest, skinfriction can have any value between \( -W \) and \( +W \), depending on the values of \( F^1 \) and \( F^2 \), as can be seen in fig.17.

Fig.17 is a further simplification of former fig.11 that has proved to lead to acceptable results.

The above has demonstrated that even in cases where skinfriction comes into the
picture the simple wave equation still holds true, using the results of the frictionless case.
There is no need whatsoever to use a (numerical) finite difference method nor a finite element method.
Moreover, the proposed solution has a number of important advantages;
- for any given level x in the pile (or hammer, or anvil, etc.) always the simultaneous stress intensity \( F(x,t) \) and velocity \( v(x,t) \) are known and not the velocity at centers of gravity of the elements with the stress somewhere in the spring between adjacent centers of gravity;
- phenomena at discontinuities of the pile (or any other part in the system), viz. the reflections and refractions can now be evaluated in a correct manner;
- phenomena occurring in any place in the system where connections do not allow traction can now correctly be assessed, such as gravity connectors, dropright-anvil and anvil-pile situations. And for the Hydroblok hammer the impacthead seating situation inside the "drop-weight". As soon as the contact force reaches zero value with a tendency to become negative (traction), the HBM-computer program regards both ends to act as free ends and further waves are reflected with negative sign. Widths of gaps can be calculated as a function of time and consequently the moment when such parts regain their contact again, the normal process is restored;
- columns of gas and hydraulic fluid can be taken into full account.

It can be understood now why most pile-driving programs are unsuitable to process correctly a Hydroblok hammer and it explains the many erroneous output results.

**HBM-PILEDRIVING COMPUTER PROGRAMS**

The theory outlined above made it possible to design a system of computer programs which are currently in operation since a couple of years.
Because of the deeper understanding so achieved there are next to the Hydroblok version also versions for conventional steam and diesel hammers, all based on the same principles.

The HBM-computer programs calculate the wave intensities \( (F_{1+4+11}) \) and the velocities \( (v_{1+4+11}) \) for a number of levels along the pile, as well as in the anvil and in the hammer for instants \( t \) equal to an integer number of timesteps \( \Delta t \).

The program selects this timestep (approx. 0.05 milliseconds) in such a way that all parts of the hammer comprise an integer number of distances \( \Delta L=\Delta t \) (for steel in the order of 0.23 m, for concrete 0.20 m).
The lengths of the pile parts are adjusted to integer multiples of \( \Delta L \).
The whole system of hammer, anvil and pile so determines a suitable coordinate system, both in distance as well as in time.
The program calculates forces and velocities for the "grid points".
The (continuous) skinfriction is split up in concentrated frictions acting in the grid points along the pile; the program performs this in such a way that the base values \( W_0 \) of the concentrated frictions are statically equivalent to the base value of the continuous skinfriction.

**COMPUTER - INPUT AND OUTPUT VALUES**

The HBM-piledriving program requires the following input data:

- **Hammer**: weight, dimensions and material of all parts, impact velocity of the dropright, bufferforce value (for Hydroblok).
- **Anvil**: weight, dimensions and material of anvil-pile situations.
- **Cushion**: no cushion with steel piles.
- **Pile**: lengths, cross sections and material.
- **Soil**: magnitude and distribution of the basic values of continuous skinfriction, overall damping factor, pointresistance, its initial value, ultimate level and time of increase.
- **Optional**: if plots are required to be stated which kind (forces, velocities, displacements) and at which level along the pile.

The following output information is received:

- Repeat of the input values.
- Energy at moment of impact.
- For a number of instants (at regular intervals) the displacements of top and toe and the energy (kinetic energy and stress energy) in hammer, anvil and pile, the energy consumed by the skinfriction and by the pointresistance. These energies are calculated from the calculated velocities and stresses and serve as a check on a correct performance of the program.
- In case of rebounce the time and the velocity of the hammer.
- Maximum and minimum stress in each distinct part of the pile.
- Optional: force-time (impact-) diagrams and velocity-time diagrams. Plotting takes place with a separate plot-program.
The computer analysis results show to be amazingly realistic. Repeatedly it was proved how correctly processes were analysed during driving with a Hydroblok, especially in unexpected situations. Fig. 18 is such an example showing the measured force-time diagram (strai gauge) next to the computed one.

**PILEDRIVING THEORY APPLIED**

Next to the purpose of predicting drive-ability prior to driving it is the proper evaluation of actual pile driving results that is of interest. For shortness sake only the latter can be discussed here.

Based on the evaluation and interpretation of numerous test results derived from instrumented piles and the concurrent use of the developed computer program it could be proved repeatedly that dynamic soil resistances and their distribution could even be determined from a non-instrumented pile, driven by a Hydroblok.

Granted that it is a laborious way to do requiring quite a number of computerruns, but it can be done (4).

Shortly summarized the principle is as follows.

During driving the complete blowcount diagram must be made and impact velocities (of the "drop weight") must be carefully measured (not guessed!). The analysis starts with assumed values for the soil resistances and an assumed distribution; the availability of a soil mechanics report of course is a great help. By numerous iterations the computerruns are repeated with each time a slight soil input modification until finally the computed blowcount diagram corresponds with the measured one. Close correspondence in this case can only mean a close approximation of the real resistances, as could be proved with instrumented piles.

As said before an instrumented pile unveils much quicker and easier the soil resistances. It will be shown here how for instance direct measurement of point resistance and skin friction near the pile tip has been performed.

In this case the pile must be instrumented with strai gauges at two or three levels about one piles diameter from each other and from the pile toe. Though the method is valid for any hammer, direct measurement is only possible with the Hydroblok due to its fairly constant force level during impact. This is explained in fig. 19 where propagation of forces is depicted.

\[ F_2 = F_1 - \frac{1}{2} W \]
\[ F_3 = F_2 - \frac{1}{2} W_2 = F_1 - \frac{1}{2} (W_1 + W_2) \]
\[ F_5 = F_4 - \frac{1}{2} W_1 \]
\[ F_3 = W_p - F_3 = W_p - F_1 + \frac{1}{2} (W_1 + W_2) \]
\[ F_5 = F_3 + \frac{1}{2} W_2 = W_p - F_1 + \frac{1}{2} W_1 + W_2 \]
\[ F_6 = F_5 + \frac{1}{2} W_1 = W_p - F_1 + W_1 + W_2 \]

Strai gauge A at time \( t_6 \) shows a total force \[ F_6 = W_p - F_1 + F_4 \]

Strai gauge B at time \( t_5 \) shows a total force \[ F_5 = W_p + W_2 - F_1 + F_4 \]

\[ \Delta F = W_1 + F_6 - F_4 \]

Only for a Hydroblok \( F_6 \) and \( F_4 \) are nearly equal. They are the result of the hammer impact with a time interval \( t_6 - t_4 = \frac{C}{a} \).

For \( a = 1 \) m and \( C = 5000 \) m/sec. (steel) \( \frac{C}{a} = 0.20 \) milliseconds, well within the Hydroblok range.

Hydroblok's impact force being nearly constant \( F_6 - F_4 \approx 0 \) and \( \Delta F = W_1 \), skin friction acting between A and B.

Assuming further that equally \( F_1 - F_4 \approx 0 \) and that unit skin friction between pile toe and B equals the skin friction between A and B the point resistance \( W_p \) can be found by simple extrapolation.

The damping influence may be neglected here provided that only strai gauge outputvalues are taken near the end of the hammer impact, when the pile tip is in motion with only a very small velocity.

Because instrumentation near the pile toe poses many practical problems also a solution has been worked out that enables the same results to be achieved with a minimum of instrumentation located only near the top of the pile. There is one restriction; the pile must be in one piece.

Strai gauge(s) and accelerometer(s) connected at a certain level in the upper part of the pile that must always remain above ground-level allow downward and upward waves to be separately measured, because:
\[ F(x,t) = F_{\downarrow}(x,t) + F_{\uparrow}(x,t) \]
and
\[ v(x,t) = \frac{F_{\downarrow}(x,t) - F_{\uparrow}(x,t)}{L} \]
(obtained by integration of the accelerometer output)

leading to
\[ F_{\downarrow}(x,t) = \frac{1}{2}(F(x,t) + 2v(x,t)) \]
and
\[ F_{\uparrow}(x,t) = \frac{1}{2}(F(x,t) - 2v(x,t)) . \]

In the wave-path diagram of fig. 20 it is shown that the upward wave carries information about the resistances. Each downward wave travelling over a distance \( dx \) decreases by an amount \( \frac{1}{2}w \) as well as each upward wave increases by an amount \( \frac{1}{2}w \).

An upward wave along the path X-Y therefore increases from \( F_{\uparrow x} \) to \( F_{\uparrow y} = F_{\uparrow x} + \frac{1}{2}W(x) \).

If X is on the first downward wave-path A-B-X-P,
\[ F_{\downarrow x} = 0 \] and \( F_{\downarrow y} = \frac{1}{2}W(x) \).

Similarly can be found for wave-path \( P'-Q' \) (\( P' \) being just above toe level on the first downward wave);
\[ F_{\downarrow Q'} = \frac{1}{2}W_{tot} \]

Immediately afterwards (path P-Q) the upward wave starts with the reflection of the first downward wave at the pile toe;
\[ F_{\downarrow p} = W_{p} - F_{\downarrow p} \]

Now \( F_{\downarrow p} \) is equal to \( F_{\downarrow A} = F_{\downarrow B} = \frac{1}{2}W_{tot} \) and consequently
\[ F_{\downarrow p} = W_{p} - F_{\downarrow A} + \frac{1}{2}W_{tot} \]

Travelling from P to Q it increases again by \( \frac{1}{2}W_{tot} \), so
\[ F_{\downarrow} = W_{p} + \frac{1}{2}W_{tot} - F_{\downarrow A} \]
\[ F_{\downarrow A} + F_{\downarrow Q} = W_{p} + W_{tot} . \]

In more general form this can be written as
\[ F_{\downarrow}(t) + F_{\uparrow}(t) = W_{p} + W_{tot} . \]

Assuming the impact force starts at the top of the pile (length \( L \)) at \( t=0 \) it is found that for

1st period;
\[ 0 \leq \frac{L+H-2D}{c} \]
\[ F_{\uparrow} = 0 \]

2nd period;
\[ \frac{L+H-2D}{c} \leq \frac{L+H}{c} \]
\[ F_{\uparrow} = \frac{1}{2}W(x) \], increasing from 0 to \( \frac{1}{2}W_{tot} \).

3rd period;
\[ \frac{L+H}{c} \leq t \leq \frac{L+H}{c} \]
\[ F_{\uparrow}(t) = \frac{1}{2}W_{p} + W_{tot} . \]

\( t_{s} \) = duration of impact.

Similar results have been found independently by Goble (7).

In the above damping is neglected for simplicity reasons. If damping is correctly assessed the following will be found for the 2nd period;
\[ F_{\uparrow} = \frac{H-D}{c} \int_{0}^{(H-D+x)} W(y-H+D)(1+\varphi)(y,T)dy \]

with \( T = t \).

For this so-called convolution integral may be written;
\[ F_{\uparrow}(t) = \frac{1}{2}W(x)(1+\varphi(t)) . \]

The function \( \varphi(t) \) turns out to be approximately a linear function of \( t \). It is possible to make a reasonable estimate for \( \varphi(t) \) and so for the base value of the total skin friction from ground-level to depth \( x \). It is this result that can be used as a first approximation for the input in the piledriving program, resulting in a simulation of the measurements. Comparison with the real measurements yields a second approximation for the input values. Only few such iterations generally suffice to get close enough to the resistance distribution that may be trusted to correspond with reality.

**CONTROLLED PILEDIVING**

From the foregoing it must have been made clear and it is emphasized here expressly that controlled piledriving surely requires something more than counting blows, measuring penetrations and comparing those with computer calculated predictions.
First of all the net energy really delivered to the pile must be exactly known. Measuring impact velocity therefore is an absolute must in order to ensure said comparison to be appropriate; guessing or misunderstanding efficiencies must be avoided. A measuring device for this purpose is part of the standard Hydrobloc outfit.

Secondly adjustment of energy as well as impact force during driving is highly desirable. Hydrobloc hammers allow for both independently in the range of 1:5. Especially the property to change the bufferforce (impact force, driving force) without affecting the impact velocity or vice versa makes it possible to govern the shape of the impact-diagram from small and tall to large and low (fig.21). This possibility to accommodate to changing soil resistances enables minimum driving times to be reached by getting more profit out of delivered energy avoiding unacceptable stress peaks to develop in the pile.

Thirdly it is the comparison of known impact-diagram and the computer simulated one, based on assumed (or measured) soil parameters that have improved the quality of drivability predictions. While driving it is the "horizontal" part of the Hydrobloc's impact-diagram combined with the M2M-computer program that enables reliable figures of encountered dynamic resistances to be produced.

It is beyond the scope of this contribution to stop into details, but in the case of concrete piles precise prescriptions, sometimes in the form of a graph are provided stating the acceptable "drivability window". It shows the range of combinations of impact velocity, bufferforce and penetration per blow, between which the operator may perform his driving safely. It assures that stresses will remain below specified limit values.

**DYNAMIC RESISTANCES MEASURED DURING DRIVING**

In the paragraph "Pile driving Theory Applied" three basic possibilities to arrive at the desired soil information were mentioned:
- extensive instrumentation of the pile;
- near the pile toe, the pile top and in between,
- minimum instrumentation of the pile; only near the pile top,
- no instrumentation at all of the pile.

These methods are extensively described in the preceding paragraphs to clarify that the results of all actions and reactions (i.e. hammer and soil) at any level in the pile translated into forces, velocities and displacements can be measured as well as computed. If computer simulation leads to correct penetrations, correct duration of impact and nearly correctly shaped force versus time diagrams (respectively velocity diagrams or displacement diagrams) at any chosen point of the pile it must be clear that the input values of soil resistances of the last computerrun are near to reality. If these comparisons are repeated at different levels during the driving operation it stands to logic that dynamic resistances are unveiled. In other words, dynamic resistances are "measured during driving". Accuracy and distinction of point resistance and skin friction obviously is best with the extensively instrumented pile, still quite acceptable with the minimum instrumentation and not doubtful in the case no pile instrumentation is used, but this requires many iterations and hardly permits in the field decision because of the time involved. The proof of all these dynamic processes cannot be given by a static test, but in general the coinciding values and especially the corresponding diagrams are a proof in itself that actions, reactions and mathematics are correctly applied. There still remains the gap between dynamic resistances and static bearing capacity. It should be emphasized that in principle only dynamic measurements are possible since pile driving incorporates pile movement. Usually dynamic resistances are less than static ones. Recovery of soil resistance and negative friction in onshore applications do influence real bearing capacity. Some indications about static bearing capacity can be derived by "measuring" a few extra blows after some set-up time.

It has been made clear that there now is available a tool and a method to derive accurate information from the soil during driving. The method is applicable with all sorts of piles in all kinds of situations, on land, at sea and also under water.

**LOADTEST COMPARISON**

Full scale loadtests are costly and in offshore circumstances practically impossible. With actual loadtests on shore having proved the feasibility of the above mentioned approach it is obvious that great savings and maximum safety for offshore conditions can be provided with minimum effort. When straingauges remain intact also during the loadtest, the separation can be made between endbearing and skin friction as well. A recent test will be briefly discussed here.
The pile was a closed ended steel "pipe" pile, originally 28 m long (phase 1), finally extended to 36 m (phase 2), steel section 121 sq.cm, driven with an HBM-500 type hammer. The "pile" was composed of two longitudinally welded sheet piling profiles. Offshore communication with the computer base some 50 miles away was simulated by transferring measured data as plotted impact-diagrams and hammer evidence by telephone (via Rank Xerox Telescopier). 45 minutes later the calculated dynamic resistance at the design depth was returned to the site. It showed a value that made it hard to believe that static design bearing capacity could be reached (fig.22). This conclusion was borne out by load test carried out afterwards on adjacent piles.

Calculated dynamic resistances at several levels of penetration are shown in the Cone Resistance Graph. There is reasonable correlation with cone resistance. At design depth (28 m) below datum the "measured" resistances are: end bearing 50 tons and skin friction 85 tons which makes a total resistance of 135 tons. Even additional set-up could certainly not bring the ultimate capacity to the required +270 tons. Nearby test piles showed ultimate capacities of 200 to 220 tons.

Resumed driving (after 2 months) on the extended pile was again monitored by dynamic measuring. The first blows showed a resistance of 235 tons. After some 40 to 50 blows friction decreased rapidly. Driving was continued until at 36 m below datum a dynamic resistance of 255 tons was measured. Static testing carried out on this pile at this penetration showed an ultimate capacity of about 350 tons. Distinction in end bearing and skin friction is also shown in fig.22.

In this case an extensively instrumented pile was used; as an example plots of all the strain gauges at their indicated levels are shown in fig.23.

Simple evaluation of the three bottom strain gauges gives end bearing value and skin friction (dynamic) at that moment in the driving operation. Full computer analysis is based on a set of readings at different depths fed to the computer and iteratively adjusted.

CONCLUSIONS AND RECOMMENDATIONS

As shown previously wave equation based computer programs are an adequate tool to predict driveability and to assess encountered resistances (8).

The authors believe the Hydroblok hammer and the HBM-computer program to be a contribution to a better understanding of the pile driving process and the governing thereof. Advantages of the Hydroblok hammer are discussed and its underwater control is described.

The HBM-piledriving analyses are extensively dealt with. Continuous check and recheck as well as tests have proved them to be reliable. One example is given of a performed test and its evaluation.

Instrumented piles, driven with a Hydroblok hammer, do provide fundamental knowledge about soil parameters and bearing capacities, leading to better controlled and more economic foundations.

ACKNOWLEDGMENT

The authors are glad to have the opportunity to publish the basic part of the study of piledriving phenomena performed by Voitus van Hamme.

Further contributions may be expected from him within short.

The design of the Hydroblok became possible only with the help of this theoretical background.

The authors bear gladly in mind the initiation by Professor H.C. Duyser of these piledriving studies.

REFERENCES


TABLE 1 - ENERGY PER BLOW SELFWEIGHT RATIO

<table>
<thead>
<tr>
<th>ENERGY PER BLOW</th>
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<td>797000</td>
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TABLE 2 - MAIN DATA OF 3 HYDROBLOK TYPES

<table>
<thead>
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<th>HBM 500</th>
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<td>BUFFER FORCE</td>
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<td>lbs</td>
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*LICENSED TO VEROLME ENGINEERING COMPANY LTD (MEMBER OF THE RHINE SCHELDE VEROLME GROUP) OF ROTTERDAM*
Fig. 1 - Hydraulic hammer components.

Fig. 2 - Impact-diagram for the hydraulic hammer (realistically).

Fig. 3 - Impact-diagram for the hydraulic hammer (schematically).

Fig. 4 - Principles of underwater driving.

Fig. 5 - Underwater aircontrol panel.
Fig. 6 - Hydraulic hammer in yokeframe.

Fig. 7 - Point resistance in sand-measurement results of concrete piles $\phi$ 45x50cm; at penetration 28.85m.

Fig. 8 - Static resistance-deflection diagram.

Fig. 9 - Static idealized resistance-deflection diagram.

Fig. 10 - Dynamic idealized resistance-velocity diagram.

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<tr>
<th>WEIGHTS</th>
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<td>1846.2</td>
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<tr>
<td>ABOVE WATER:</td>
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<td></td>
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<td>TOTAL WEIGHT (MINUS BALLAST)</td>
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<td>502</td>
</tr>
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<td>PARTS OF YOKE FRAME HANGING IN CRANE HOOK</td>
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<td>66.2</td>
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<tr>
<td>WEIGHT ON TOP OF PILE (WITHOUT COMPENSATION)</td>
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<td>UNDER WATER</td>
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<td>313.4</td>
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Fig. 11 - Modified dynamic idealized resistance-velocity diagram.

\[ V_{o1} \quad V_{o2} \]

WAVE IN ROD 1 + WAVE IN ROD 2 = COMMON VELOCITY \( V \)

AT IMPACT: WAVE IN ROD 1 IN NEGATIVE DIRECTION = \( F_{11} \)

\[ V_{o1} = \frac{F_{11}}{z_1} \quad V_{o2} = \frac{F_{12}}{z_2} \]

\[ V_0 = \frac{F_{11} + F_{12}}{z_1 + z_2} \]

\[ F = (V_{o1} - V_{o2}) \frac{z_1 z_2}{z_1 + z_2} \]

\[ V = \frac{V_{o1} z_1 + V_{o2} z_2}{z_1 + z_2} \]

Fig. 12 - Stress waves travelling along rod.

Fig. 13 - Impact of two rods.

Common Velocity \( V \)

Waves \( F_{11} \) and \( F_{12} \) meet at discontinuity: Waves \( F_{11} \) and \( F_{12} \) are generated:

\[ F_{11} = \frac{F_{11} - F_{12}}{z_1} \quad F_{12} = \frac{F_{11} - F_{12}}{z_2} \]

\[ F_{11} = F_{11} \times \frac{1}{z_1} \times D_{11} = \frac{1}{z_1} \quad F_{12} = F_{12} \times \frac{1}{z_2} \times D_{12} = \frac{1}{z_2} \]

R = REFLECTION FACTOR \[ \frac{z_2 - z_1}{z_2 + z_1} \]

\[ D_{11} = \frac{z_1}{z_2} + 1 \quad D_{12} = \frac{z_2}{z_1} + 1 \]

Diffraction Factors

Fig. 14 - Discontinuity in a rod.

Fig. 15 - Superposition of wave plus reflected wave (rod at right hand end fixed).

Fig. 16 - How skin friction affects the waves.

Fig. 17 - Simplified dynamic skin friction-velocity diagram.

\[ W_{\text{f}} \quad W_{\text{r}} \]

For \( v = 0 \), \( W \) can have any value between \(-W_{\text{f}}\) and \(+W_{\text{r}}\).
Fig. 18 - How computer force-time diagram coincides with measured diagram.

Fig. 19 - Strain gauge measurements near pile toe.

Fig. 20 - Wave-path diagram.

Fig. 21 - Two different forces, same amount of energy.
Fig. 22 - Example of calculating dynamic resistances during driving.
Pile tip at 35.40 m below datum (32 m penetration).

Impact velocity 5.3 m/sec.

Buffer force 150 kN ($F_b$)

Penetration per blow 0.65 cm.

Fig. 23 - Strain gauge and displacement readings of piledriving process shown on Fig. 22.
NORTH SEA PILE DRIVING EXPERIENCE WITH A HYDRAULIC HAMMER

by Joost W. Jansz, Hollandsche Beton Groep N. V.

ABSTRACT.

This report presents field operational results and experiences on a pilerediving project in the North Sea during summer 1976; theories nor calculations will be given here.

Two HBM-3000 type HYDROBLOK offshore hammeres have driven the 60" O.D. skirtpipes in the 12,000 ton steel jacket of Occidental's Claymore field production platform.

High number of blows-per-minute and high impact efficiency of each blow have resulted in unusually short driving times. Ease of handling due to new constructional items are described, such as the pile sleeve and the flat bottom anvil that does not contain cushion blocks.

Apart from being thoroughly tested during larger periods of time on a vertical test-pile, it is the real driving of 19 batter piles down into the bottom of the North Sea in a real offshore operation that gives evidence of the reliability of this modern type of hammer.

Remote controllability of all hammer functions has been exercised, being of great importance with the view on further n.r future applications. The Claymore field pilerediving took place from above water with the aid of followers.

This year the same type of hammer is going to be applied in a fully operational application in more than 1000 ft. waterdepth.

INTRODUCTION.

Earlier presentations have described Hydrobloc hammeres in detail with respect to their specific characteristics, their construction (1,5, 11) as well as to the wave theory based computer programs PILEWAVE (2,5, 10) that have formed the scientific isis for the constructional concept.

REFERENCES AND ILLUSTRATIONS AT END OF PAPER.

Shortly summarised the design philosophy behind this hydraulically driven hammer is that at each blow there are two physical properties that are clearly distinguished: FORCE (in lbs, kips or tons) and ENERGY (in ft. Ib or meters. tons). Energy can be kept constant while the operator can vary the force that at each blow is applied to the anvil at the moment of impact. It is amazing to observe how quickly an experienced operator finds out that it is not at all times the highest force that makes his pile penetrates faster. Sometimes it is the longer time duration that a lower force can be kept active on top of the pile that renders a greater penetration. For this reason this hammer is equipped with a built-in "buffer" of patented construction.

But the "buffer" serves two more important purposes. The first being that there is no need to have any cushioning material of whatever nature in the anvil. Direct steel to steel contact at the moment of impact has now become fully acceptable; no uncontrollable peak forces can come into existence neither in the dropweight, nor in the anvil and also not in the pile. Impact piledriving forces therefore are transferred into the pile in the most effective sense possible, provided all steel surfaces do really bear squarely; the patented pile sleeve construction in which the anvil is captively incorporated ensures this (fig. 1). The anvil itself (fig. 2) can be of the most simple and appropriate form affording the best high tensile quality steel to be chosen for longest lifetime. Its flat bottom matches every pile size without any adaptation.

The function of guiding the pile is fully taken care of by the aforementioned pile sleeve (fig. 3); to match other pile diameters the pile sleeve can easily be adapted.
It has been measured many times that in all circumstances at random fall forces really cannot be generated by this hammer; and this is the third purpose that the "buffer" serves. It protects the hammer construction itself very efficiently from excessive forces and stresses that normally must lead to damage of hammer-parts. Practical evidence that this really holds true has also been given in the North Sea where the hammer had to sustain continuous uninterrupted driving during long periods of time with blowcounts of 1610 blows per foot, requiring about half an hour continuous hard driving. On a permanently available 84" O.D. test pile (fig.4) each HBM-3000 type hammer is tested before delivery; blowcounts of as much as 5000 blows per foot have already been recorded.

SITE CONDITIONS.

Claymore field in the UK North Sea Block 14/19 has a 360 ft. waterdepth. The platform is a 8-legged platform with 6 skirt piles around each main leg and 4 pinpiles between the legs. The skirt piles at a batter of 1.125:12 in one direction are 60" O.D. with wall thickness varying from 2 to 3½ inch. Final design penetration was 150 ft. Pile lengths were 300 ft. The driving took place above water on top of followers of the gravity connector type. It was Netherlands Offshore Company (NOC) who operated its two HBM-3000 Hydroblok hammers from Heerema's derrick barge "Challenger". Two Menck MRBS 8000 steamhammers of Heerema's were also used from the same ship. Two more MRBS 8000 steamhammers were operated from Brown and Root's barge "Hercules", located at the opposite side of the jacket. The pile driving operation started June 26, 1976 and was completed in little more than three weeks on July 19, 1976. From the total of twenty four skirt piles and four 42" pin piles, there were 19 and 2 respectively driven by the Hydroblok hammers. Generally speaking the driving was heavy and at times too heavy for the steamhammers facing too high blow counts. Under these circumstances the Hydroblok hammer property that high blowcounts are acceptable proved to be of utmost importance.

The weather during the operation did pose little problems. In the next paragraph the behaviour and performances of both types of hammers are compared:

- the steamhammer with its conventional offshore cage.
- the Hydroblok hammer with its pile sleeve.

HAMMER PERFORMANCES.

Manufacturers always state the energy-performance of their hammers as "rated energies". With the free falling dropweight types this simply represents the product of mass and stroke. For analysis purposes an efficiency factor is introduced that must be applied to this rated energy value. The assessment of a realistic efficiency factor for a certain case however can only be a rough guess. Exact figures are not available. It is known from the few measurements ever been made with heavy offshore hammers that the remaining nett energy that effectively is delivered to the pile top showed to be disappointingly low.

The construction of conventional steamhammers of open cage construction, their guidance, the capping material, the poor alignment not always providing the expected square bearing of the various parts are unfavourable at random factors for a high efficiency rating. In the case of a Hydroblok hammer the fully enclosed and extremely stiff construction provides perfect guiding of the accurately machined parts under ideal lubrication conditions. Table 1 shows how the ultimate efficiencies compare between the two types of hammers.

Besides a device is built-in that automatically measures the speed of the dropweight at the moment of impact; the operator knows at each blow what his impact speed really is enabling him to increase if necessary either his energy and/or the impact force.

Shortly after impact the dropweight starts its upward movement. This speed also is measured automatically. With the simple formula of \( \frac{1}{2} M (V_1^2 - V_2^2) \), where \( V_1 \) = downward impact speed and \( V_2 \) = upward return speed, shows the nett energy that the pile really has accepted. Table 2 shows an arbitrary example. The operator now for the first time has been given the possibility to continuously adjust his driving to changing situations. In case he notices that his return upward speed has a tendency to increase he knows that the pile accepts less energy and he can now improve this by increasing his "buffer"-force. At the Claymore field piling driving this has been proved to work well. And this is an extremely important feature of the hammer as soon as it is going to be used under water. Not having any audible or visual contact with the hammer, the operator has now been given a reliable tool to "see" his driving remotely and to correct accordingly. Said values can easily be recorded.
Though the Menck MRBS-8000 steamhammer is rated to be the more powerful hammer it was shown at the site that the HBM-3000 Hydroblok hammer was able to overcome greater driving resistances; a good 10% was felt to be the difference, at lower blowcounts even higher. The table with no. 3 shows some results. A considerable spread of final blowcounts was noted among piles, even in the same cluster. One skirt pile (A5-1) was driven both by the Hydroblok and the steamhammer showing the tendency that blowcounts of the steamhammer remain higher. One should however be careful in making too simple comparisons. The possibility however that any blowcount-figure in principle is acceptable for the Hydroblok hammer has clearly proved to be of great importance in those typical North Sea overconsolidated soil layers.

BATTER PILES - PILE SLEEVE.

All skirt piles had a 1:125:12 batter in one direction. Fig. 5 shows the conventional offshore cage being kept supported in the crane. The steamhammer rests on the pile during driving. In the case of MRBS-8000 the hammer and anvil weight on the pile is 192 tons (of 1000 kg.), the 85 tons weight of the offshore frame hangs in the crane. This conventional solution has the disadvantage that the crane ship with its boom over the side starts easily to roll thereby introducing undesired influence of the hammer in the cage. At times the roll of the ship, in during calm sea, became so rough that the pile driving had to be interrupted for quite some time.

Fig. 6 shows the Hydroblok hammer sitting freely on top of the pile. Its sleeve acts as a perfect guide for the pile, its internal diameter matching the pile 0.5. The hammer casing rests on top of the anvil inside the pile sleeve keeping the anvil firmly pressed on top of the pile. At each blow the hammer thereupon must always hit centrally and squarely. From fig. 9 it can clearly be seen that the hammer does not need any other support than the pile itself. As soon as the hammer has been placed on top of the pile, the driving can be started. The hydraulic drive allows the operator to start carefully, giving only one blow at a time with low energy and low force or, when the situation permits, with high energy and low force. At all times he keeps his driving under perfect control. If he wants to give just one blow, he gives one blow without one or more after blows. As a matter of fact the -batter- pile must have sufficient stiffness to bear the 180 tons weight of the HBM-3000. Every cantilever batter-pile will bend under the hammer weight; what happens when this bent pile is given a hammer blow of thousands of force?

Will it buckle, will its bending increase, will the pile plus hammer start to sway and will it perhaps come into the dangerous region of its own frequency?

Practice has clearly shown that nothing happens; the pile remains perfectly calm, its only movement being the penetration under influence of each hammer blow. Theoretical expectations were confirmed. The strongly dynamic 3000 tons impact force of each hammer blow initiates a force wave to travel through the pile of such a short-duration that it can not create bending, nor buckling; both these physical phenomena require a minimum time duration in which the force should remain active which in the case of traditional pile driving is not available. Of course the stresses in the pile due to the 180 tons static weight of the hammer must be acceptable. In the case of the Claymore field the situation was as schematically shown in fig. 7. Under continuous hard driving the pile-hammer combination remained perfectly quiet.

HANDLING: HAMMER, HOSES.

There is no need to replace the cushion block every 3000-5000 blows; as referred to previously the hammer does not require any cushion block. The pile sleeve not only provides the hammer with a perfect guide for the pile, but it also facilitates the entire hammer handling from its horizontal storage position on deck on to its near vertical position on top of the pile. As soon as the hammer is seated on the pile, the crane hook is lowered to slacken the slings and the driving can immediately be started and this can be continued irrespective of barge movement. The pile sleeve needs a shorter pile length above the jacket when compared to the offshore cage in case a steamhammer is used.

The hydraulic hoses were thoroughly tested prior to being used on the North Sea. No problems were therefore anticipated regarding their quality, but no experience yet was available with respect to their handling under operational conditions. Compared with steamhoses, generally connected at one end to the hammer and at the other end to the steam line fixed in the craneboom, the 3 hydraulic hoses of the hydraulic hammer were even less an obstacle. Especially the fact that these hydraulic hoses were connected directly from the power pack on the deck to the hammer, turned out to be an ideal solution, basically for two reasons. As soon as the hammer was back on deck again, the crane was free to be used for any other purpose; no disconnection as is usual with steamhoses. Secondly the hydraulic hoses need not to be disconnected at all, leaving the entire oil circuit closed, without leakage. It requires about 20 minutes to hoist the HBM-3000 hammer from its horizontal position on deck and set it on a pile, or vice versa.
The philosophy not to bring the hydraulic hoses in the craneboom proved to be correct, when this handling time is compared with the steambammer requiiring about 2 hours crane time to piece up the case that steam lines require connecting. In case a hydraulic hammer must be taken from the pile to be replaced by the back up hammer, it takes about 2 hours including the shifting of the hydraulic hoses, before the other hammer is in operation on the pile again.

**DAMAGES, INCONVENIENCES.**

The two hammers remained serviceable, though a few troubles were encountered. The most troublesome was the breakdowm internally of an auxiliary valve in the hydraulic steering system of one of the hammers; the broken parts travelled uncontrolled through part of the hydraulic system, without causing damage and also not causing the hammer to break down. The damage, as inspection has learned was due to cavitation because the broken parts had blocked partwise certain hydraulic passages. Before completion of the project one hammer was sent to shore for inspection, because it was expected that hammer no. 2 would be able to complete the job, which turned out to be correct. These auxiliary valves were expected to be necessary to avoid unacceptable heat-concentration in a certain part of the system. Thorough testing and temperature measurement in a later stage have learned that those auxiliary valves could be left out completely. Complete disassembly and inspection of both hammers when the driving was completed have shown no significant wear nor any other defects. Once more it had become clear, that also for these huge hammers the specific Hydroblok principles are correct with respect to the driving as well as to the hammer design.

There were a number of minor inconveniences such as loosened hose clamps, bolts that had to be locked and secured in a more efficient manner. There seemed to be a lubricating problem with one hammer, which however turned out not to be so, but instead was a cracked metal surface where no oil film could be permanently built on. For one reason or another these cracks resulting from incorrect heat treatment passed inspection without being discovered.

So far the conclusion seems to be justified that there are no fundamental errors or untraceable barriers. The problems encountered could be well understood and explained, and their causes are known; essential conditions for a designer, manufacturer and offshore contractor in their aim to have the disposal of a reliable piece of equipment.

**FUTURE, UNDERWATER DRIVING.**

It is obvious that the described field operational experiences are an important milestone in this recent Hydroblok development. Any further use above water is not expected to pose new problems. Operating crews have acquainted themselves with these new hammers, maintenance personnel now is familiar with the interior of these completely closed hammers. They know that they may rely on these pieces of real machinery, provided they follow carefully the instruction manuals. These piledrivers are comparable to a combustion engine as far as maintenance is concerned. Preventive maintenance must make also this hydraulic hammer a highly reliable piece of equipment.

Amongst other aspects and demands referred to earlier in this contribution, it is this reliability that is required before the hammer can be used successfully in greater water depths.

At the time this paper was written another two HBM-3000 Hydroblok hammers fully tested on the testpile (above water) are ready to be put into regular operation in water depth up to 1300 ft. Not only the hammer with its 3000 hp. dieseldriven hydraulic power pack but also an extensive range of annex equipment has been supplied to perform this deep water heavy piledriving in a fully reliable and controlled manner. It is hoped that soon details can be reported and be made public.

**ACKNOWLEDGMENT.**

The author likes to emphasize that the realisation and the materialisation of the Hydroblok Piledriving Hammer Systems are the result of a multidisciplinary effort of a group of enthusiastic technicians co-operating closely in their aim to contribute to the solution of a technical problem. This research is done within the Hollandsche Beton Groep N.V. of Rijswijk, the Netherlends. The right to manufacture, market and sell has been licenced to Verolme Engineering Company Ltd., member of the Rhine Scheide Verolme (RSV) Groep of Rotterdam, the Netherlands.

**REFERENCES.**


7. Begemann, dr.ir. H.K.S.Ph., "The Dutch Static Penetration Test with the Adhesion jacket Cone", LGM Mededelingen (XII, No. 4, April 69), 69-99.
8. Goble, G.G., "Bearing Capacity of Piles from Dynamic Measurements", Department of Civil Engineering, Case Western Reserve University, Cleveland, Ohio (1975) 1, 77.
### TABLE 1 - ULTIMATE EFFICIENCIES COMPARISON

<table>
<thead>
<tr>
<th></th>
<th>Steamhammer</th>
<th>Hydroblok</th>
</tr>
</thead>
<tbody>
<tr>
<td>Energy according to prospect velocity measured at moment of impact</td>
<td>120 t.m.</td>
<td>100 t.m.</td>
</tr>
<tr>
<td>Delivered kinetic energy</td>
<td>4.7 m/sec. 88 t.m.</td>
<td>5.3 m/sec. 97 t.m.</td>
</tr>
<tr>
<td>Energy loss in pilecap (guess) (steel + wood)</td>
<td>weight 42 t. loss 20% = 17.6 t.m. not adjustable; loss (guess) 5% = 4 t.m.</td>
<td>weight 14 t. loss 10% = 9.7 t.m. adjustable, no loss due to excess of yield strength 87.3 t.m. 90%</td>
</tr>
<tr>
<td>Impact force</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Remaining nett energy Driving efficiency</td>
<td>66.4 t.m. 55%</td>
<td></td>
</tr>
</tbody>
</table>

### TABLE 2 - ARBITRARY EXAMPLE OF ENERGY

**EXAMPLE - HOW ENERGIES ARE DEALT WITH.**

\[ E = \frac{1}{2} m v^2 \]

- \( m \) = mass of ram = known. (= 69 ton)
- \( v \) = velocity = measured + printed.

**AT IMPACT;**

- Impact speed \( v_t = \frac{500}{114} \approx 4.4 \text{ m/sec.} \)
- \( E_{net} = \frac{1}{2} \times 69 \times 4.4^2 \approx 67 \text{ t.m.} \)

**AT RETURN;**

- Return speed \( v_r = \frac{500}{396} \approx 1.26 \text{ m/sec.} \)
- \( E_{return} = \frac{1}{2} \times 69 \times 1.26^2 \approx 5.5 \text{ t.m.} \)

**Thus ENERGY ABSORBED BY THE PILE = 67 - 5.5 = 61.5 t.m.**

\((485,000 - 40,000 = 445,000 \text{ ft.lbs})\)

**HBM - 3000A; MAXIMUM NET ENERGY = 110 t.m. (797,000 ft.lbs)**

"RATED" ENERGY = 1 million ft.lbs

### TABLE 3 - HAMMER DATA

<table>
<thead>
<tr>
<th>Pile Number</th>
<th>Hammer</th>
<th>Penetration</th>
<th>Blowcount</th>
<th>Total net driving time min.</th>
<th>Total blows</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>max</td>
<td>average last 5 ft.</td>
<td></td>
</tr>
<tr>
<td>A5 - 4</td>
<td>Hydroblok</td>
<td>26 - 150</td>
<td>65</td>
<td>34</td>
<td>40</td>
</tr>
<tr>
<td>A5 - 6</td>
<td>Hydroblok</td>
<td>30 - 148</td>
<td>99</td>
<td>58</td>
<td>46</td>
</tr>
<tr>
<td>A5 - 1</td>
<td>Hydroblok</td>
<td>29 - 77</td>
<td>15</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>A5 - 1</td>
<td>Steam</td>
<td>78 - 150</td>
<td>79</td>
<td>22</td>
<td>20</td>
</tr>
<tr>
<td>B1 - 2</td>
<td>Steam</td>
<td>24 - 134</td>
<td>329</td>
<td>245</td>
<td>241</td>
</tr>
<tr>
<td>B1 - 6</td>
<td>Steam</td>
<td>24 - 147</td>
<td>98</td>
<td>39</td>
<td>33</td>
</tr>
</tbody>
</table>
Fig. 1 - Hammer with pile sleeve.

Fig. 2 - Anvil.

Fig. 3 - Pile sleeve guiding the pile.

Fig. 4 - HBM-3000 type hammer on 84" O.D. testpile.
Fig. 5 - Conventional offshore cage.

Fig. 6 - HBM-3000 hammer sitting freely on top of pile.

Fig. 7 - HBM-3000 hammer sitting freely on top of pile; driving without further support.
UNDERWATER PILEDRIVING.
TODAYS EXPERIENCES AND WHAT IS ABOUT TO COME

Joost W. Jansz
HBG, The Netherlands

Summary

In late 1974 the first offshore pile was driven underwater in the Gulf of Mexico in 92 m water depth. A 0.6 m O.D. pile was given its 76 m penetration into the seabed. It was the very first real underwater hammer, seated immediately on top of a pile without any intermediate follower.

Driving took place completely submerged and full remote controllability was proven. It was a Hydrobloc hammer that set this world record.

Subsequently in 1977 the heavy 190 m long 2.13 m O.D. piles for Shell Oil's Cognac Deepwater Production Platform were very successfully driven in record time to their 152 m penetration in 320 m deep water of the Gulf. Pile and hammer-handling were of a highly sophisticated nature.

At about the same time another Hydrobloc hammer drove 1.22 m O.D. piles in the 357 m deep South China Sea. Handling here was simplified as much as possible.

The projects, given here as practical illustrations of two very different set-ups had one thing in common; there already was a structure on the seafloor prior to driving that provided 22 piles with their lateral stability.

Is it possible at all to drive a pile underwater without such a template? The answer is yes, the "Puppet"-system; this contribution deals with this system in particular. In late 1978 the first laterally unsupported free standing piles were driven in the North Sea; their purpose is to serve as subsea anchor piles.

Looking into the immediate future, the scene shows quite a number of interesting new developments such as driving conductor piles in early production systems, batter platform derris piles not requiring followers, nor guide bells, anchor piles in waterdepths of 1000 meters, etc.
Although this contribution focusses on underwater piledriving, a good understanding of its prior development is thought to be useful. Publications in the past contain most of this knowledge already in some scattered form. Here, however, an effort will be made to show how logical the developments really were that have ultimately led to our abilities today. It could very well be that corresponding logic may as well stretch further into the future.

Driving piles is a very old technique. Dropping the ramweight on top of a pile has two major aspects:

- the top of the pile experiences a force (tons) during impact, and
- energy of the ramweight (meter-tons) is transferred into the pile to be converted into pile penetration.

Pile penetration means that the soil near the pile toe must be given plastic deformation. Dimensions and material of the pile also have their specific influence on the driving result. It is evidently not only the hammer that governs the process, it is the interaction between hammer, pile and soil immediately following the moment of impact.

Decreasing the dropweight of a drophammer not only reduces the energy of the hammer, but also results in a reduced impact force, so energy and force are interdependent in that case.

Research of these phenomena has given birth to the Hydroblok-principle, where a specially designed pretensioned member is interposed between the ramweight and the pile (ref. 1 and 2). This member can either be built in the ram or in the anvil. The patented Hydroblok buffer can be remotely controlled between a minimum and a maximum force, the force being available immediately at moment of impact. It is called the "impact force" and its value in tons or kips can be read or recorded on the control panel of the operator.

It is this Hydroblok buffer that makes energy and force independent of each other, with the effect that the impact force-value can be changed while the impact energy level remains constant. The lower the impact force is chosen, the longer the period of time will be that this force remains active on the pile. And it has been found that in many cases it is the longer duration that leads to greater penetration. This is the clue which explains the short driving times of Hydroblok hammers. (ref. 3 and 4). In addition the larger number of blows per minute when compared with other hammers also adds to the high performance.

Table I shows the calculated results made for a particular case. For a given pile in a certain soil condition, the penetration result was analysed for various impact forces, all with the same impact energy. It is interesting to see that it is not always the highest impact force that must render maximum penetration.

Fig. 1 shows the calculated impact-force-time-diagrams and also the measured impact-force-time-diagrams. The calculations were performed for the Hydroblok HBM 4000 in the design stage before the hammer was built. The measurements were taken later during testing of one of these hammers on a pile of 2.13 m diameter, 32 mm wall thickness as part of the commission-procedure to the client. Comparison of the diagrams shows that the mechanical model that is adopted for the design of hammers is close to reality. This proves that hammers can be designed to serve a particular well-defined purpose. In the case of the HBM 4000 the design diagrams have been sent to a number of non-partial representatives of oil companies and certifying organizations, who sponsored part of the tests, before these hammers were built and prior to the actual tests. Fig. 2 shows the HBM 4000 on a 1:5 batter pile.
WHY HAS UNDERWATER PILEDRIVING BECOME POSSIBLE?

Though the objective of the Hydroblok research has originally been the achievement of a better piledriving performance, its side-effects have been many, some evidently opening wide perspectives to its future use underwater (ref. 5 and 6). In this respect the following aspects should be mentioned here.

- Because of the built-in buffer (Fig. 3) an all-steel anvil could be introduced to transfer the energy and the force in a steel-to-steel manner from the ram into the pile, not requiring any capfilling material. Apart from considerable losses in such material which have a negative effect on performance, its replacement at regular intervals prevents such a hammer being used as an underwater tool.

- The built-in buffer protects the hammer from excessive stress and strain. At the moment of impact shock-waves start to travel not only in the pile, but also in the hammer. Now peakforces are controlled by the buffer and consequently kept within acceptable limits. This makes it possible to design the various hammer parts properly for fatigue. Extensive stress and strain calculations have been performed, both statically and dynamically, the latter being based on known impact speeds of the various colliding hammerparts. Extensive test measurements during actual piledriving have proved that these complex calculations correctly assess what happens in practice. Only after this research-phase is over one in an acceptable way, may it be expected that the hammer is designed as a reliable tool, suitable to be used underwater during longer periods.

- The built-in buffer can be remotely controlled, its force being constantly indicated or recorded. Kinetic energy of each blow at the moment of impact is also automatically measured, displayed and recorded. Any type of operation that takes place out of sight or sound can only be mastered when certain key values that change during the operation are numerically known. This also applies to driving piles underwater.

- Another indispensable item in the underwater piledriving development is the pilesleeve, located in the lower part of the hammer (Fig. 3). In the topsection of the pilesleeve the flat-bottom anvil is captively held, the bottom generally serves as an open conical guide-bell, which continues into a cylindrical part that serves the purpose of guide onto the pile. In this way the Hydroblok hammer can be guided on the upper part of a pile, thereby securing a perfect square in-line transmission of the driving force from the ram into the pile. Also raked piles with a batter of 1:5 that have sufficient lateral support from themselves can be driven in this way; the hammer does not require any additional support or guide.

- Being seated on top of the pile the hammer can be started to drive. Built-in safety devices automatically stop the ram-movement immediately at the moment that a "no-go" situation occurs which could damage the pile-hammer system, as for instance when the pile starts to run. As soon however as the "go" situation restores, the driving of the hammer restarts automatically or can be restarted manually if preferred.

- The pilesleeve has also provided the possibility of lowering the pile with the hammer on top as one unit in one simple operation. Certain provisions prevent the hammer from losing the pile during their mutual descent. Shortly after the driving has started the temporary connection between the hammer and the pile is disconnected allowing the hammer to be retrieved. These provisions have been used for instance in the North Sea Piper Field, where heavy subsea anchor piles have been driven.

Not before all the mentioned aspects were satisfactorily solved, could one think of driving piles in greater waterdepths; this phase was reached in 1974.
CONFIDENCE in 1974 was such that the moment neared for the Hydrobloc to be baptised. On September 17, 1974, for the first time a pile was really driven underwater; an HBM 500 type Hydrobloc hammer, operating under water, made a 0.6 m diameter pile penetrate 76 m into the seabed in the Gulf of Mexico. The waterdepth was 92 m. For nine hours the hammer remained underwater, of which 5 hours actually driving, twice interrupted to remove a temporary support. Driving was terminated with 305 blows per foot (Fig. 4). This successful test operation proved that all "technical philosophies" and their materialisation were correct so far and that the underwater piledriving process could be remotely controlled. At that time all parties, amongst which were the Shell Oil Company and McDermott, were convinced that this type of hydraulic hammer was the appropriate tool to secure the largest platform ever built to the seafloor. At the same time it also became evident that much consideration still had to be given to handling procedures for so large an operation as Cognac (Fig. 5).

PILES IN SHELL OIL'S COGNAC PLATFORM

150 m long piles in one length were to be driven 150 m deep into the seabed to render ultimately more than 6500 tons safe bearing capacity each; the HBM 3000 A type Hydrobloc hammer was chosen for the purpose. We will not repeat here what has already been published in many periodicals and technical papers about this extremely interesting project, where so many new as well as sophisticated techniques were used. The piledriving turned out to be a highly successful operation, performed in record time; the last 18 out of a total of 24 of these giant piles took 21 days.

Not yet published before are the basic keys to this successful underwater pile-dri
ving in over 320 m waterdepth. Within the framework of this contribution only a few of these technical philosophies that have formed the basis of the chosen technical solutions for the various handling aspects, will be mentioned.

DESIGN PHILOSOPHY

The first step in tackling any technical problem involved is to analyse the problem in order to isolate and identify more or less self-contained partialities or sub-problems (problem areas). Maybe it is more fashionable to speak of "system-analysis" and "sub-systems". At their interfaces, the sub-systems are linked, and combined they form the complete system (which on its term always can be perceived as a sub-system of another even bigger system).

At this stage it is of crucial importance to identify clearly the one or two most critical problem areas. These must be solved first and in general little or no compromise can be accepted here. They provide the boundary conditions which will govern the solutions of the remaining problems.

Piledriving, and specially so under water, proves to be a sub-system which cannot be tackled as a self-contained problem. One has to look at the links at the interfaces of adjacent problem areas.

Let us examine Shell's Cognac Platform. The main problem, though in itself not to be underestimated, is not how a 190 m pile should be driven to a penetration of 150 m. Nor is in itself the main problem, how this should be done under water. No, on the contrary, driving heavy piles and doing so under water were problems that, in principle, were solved at that time.
The real critical problems were "How do you get the pile and hammer down?", "How do you retrieve equipment?", "How do you control and monitor the operation?". In one word HANDLING AND MONITORING; the components were there, but there was no SYSTEM.

Table II shows the problem areas and the solutions that were adopted in this particular application; one or two of these solutions will be highlighted in the following. They may serve as an illustration of the application of the adopted design philosophy.

**THE DESIGN PHILOSOPHY APPLIED ON COGNAC**

Figs. 6a and 6b show one such example. To avoid misalignment problems and bending effects of the extremely slender piles the elevator-idea of Fig. 6b was introduced. Fig. 7 shows the principle of the elevator, in this case carried by 4 cables. The elevator-lines, first used to lower the pile, serve a second purpose after the pile has been stabbed and slipped into the guidepile of the base structure; the 4 elevator lines allow the hammer to be travelled down in a fully guided fashion with a guaranteed failsafe landing on top of the pile. The earlier mentioned pile-sleeve of the hammer provides a perfect seating of the hammer. The next step is to slip the elevator down along the pile, thus allowing the driving to be started. Notice that in case of a hammer failure the hammer is easily retrievable; after repair and/or replacement it can be sent down safely again.

Introduction of the yoke frame (Fig. 8) is another example of the systems analysis outcome. During operation the hydraulic hoses, the airhose and the electric cable will run from their hose/cable-drums vertically down through the water to the yoke frame thereby being kept automatically under constant tension by the weight of the yoke frame. From the top of the yoke frame the hoses and the cable are loopwise connected to the hammer (see Figs. 9b and 9c). The 6 m stroke of the yoke frame not only compensated a certain vertical barge movement (heave) but it also gave the operator a certain stretch to drive the pile without the need to reel constantly; he had a yoke frame indicator on his control desk that showed him the position of the hammer in relation to the yoke frame. Having consumed most of the stroke the operator could reel another 6 m for the next stretch. This chosen solution resulted not only in a simple operation that could be controlled quickly, but the hoses hanging a hundred and more meters down in the water, kept under a constant tension force, were in this way only subjected to the relatively slow vertical barge movements; they were given the most ideal position at all times during every phase of the operations. Figs. 9a, b and c depict the way how in an early stage the above described "technical philosophy" was developed. Fig. 10 is a picture of what the philosophy really looked like, after it had been fully engineered.

I have described in detail a few items with their background philosophy because they illustrate how we tackled the complexity when faced for the first time with Cognac Platform underwater piledriving. How we stripped reality as much as possible from all technical complications in order that only the basic problems would remain. It is definitely not a matter of oversimplification but this technique makes the engineer focus on basic problem areas, before real technical details make him nearly drown in the engineering problems.
THE PUPPET SYSTEMS

It was the above type of technique that also rendered the Puppet Systems which make it possible to drive piles underwater without any physical support of whatever nature on the seafloor.

It has been a desire for many years to be able to drive heavy anchor piles into the seabed without the help of a template to keep the pile upright at the moment of touch-down of the pile toe and during the first hammer blows.

During two years of research, from a variety of methods two were selected for application in practice (Fig. 11):

1. A method where only lateral soil resistance is used to stabilize pile and hammer.
2. A method where a mass, the Puppet Weight, provides self-stabilization to both pile and hammer; this method works independently of soil properties.

Both methods were recently described in detail (ref. 7 and 8); a short outline will be given here.

In a Puppet System piling operation the pile and the hammer are temporarily connected for easy lowering of both as one unit. Several well-proven temporary connector systems are available today which do not need any diver assistance.

Puppet System method number 1 can only be employed if soil conditions are suitable. Its principle is very simple; the pile penetrates the soil by selfweight of pile and hammer and lateral soil resistance keeps pile and hammer upright. Theoretically, a pile could be stabbed exactly vertically using this method. In practice, however, deviations from the vertical will occur due to currents and position changes of the vessel during the pile stabbing operation. An analysis method has been developed (ref. 7) to assess the largest possible pile deviation from the vertical after stabbing. Fig. 12 shows one calculated result for Occidental's North Sea Piper Field, showing that method number 1 is acceptable. The result is the worst extreme under conditions according to table III. The graph (Fig. 12) shows that even in the worst case pile deviation from the vertical will be less than 4 degrees after stabbing. A further increase of the pile deviation during the first hammer blows may be neglected.

The soil independent Puppet System method number 2 requires two guidelines, running down from the vessel, through the puppet-eyes to the Puppet Weight (Fig. 11). The latter is simply a mass, loosely slipped around the pile and located at a low level, producing a constant tension force in the puppet-weight-guidelines. Once the weight of pile and hammer rests on the seafloor, the force in the puppet-weight-guidelines acts partly on the puppet-eyes thus stabilizing pile and hammer. The self-stabilizing capacity of this mechanical system can be clarified from the graph in Fig. 13 showing the moment of all forces on the system versus the pile's deviation from the vertical.

As long as \( \frac{dM_1}{d\alpha} < 0 \), the system will be self-stabilizing (moment \( M_1 \) and pile angle \( \alpha \) acting in the same direction).

This criterion holds as long as pile angle \( \alpha \) remains between the two Re-Stabbing points RS, then the system will always automatically return to the Equilibrium Position EP. Fig. 13 represents the worst case under conditions according to table IV. Fig. 14 shows the two area's to be noticed in practice: the self-stabilizing working area and the re-stab area's. If for some reason (e.g. an unexpected large position change of the vessel) the pile angle \( \alpha \) would come into a re-stab area, the pile and hammer must be hoisted and be re-stabbed again. The example analysed here shows that a Puppet Weight of 85 ton provides sufficient capacities to stabilize a 168 ton pile plus the piling driving hammer IBM 1500's weight of 78 ton.
FIRST APPLICATION (1978) OF THE PUPPET SYSTEM

In August and September 1978 eight anchor piles were driven into the seabed (Fig. 15) in Occidental's North Sea Piper Field. The nature of the soil at this location was as given in table III. The project was the first of this type and was conducted from the Sedco 445 drillship which is dynamically positioned. Only minor adaptations to the ship's installations were required.

Regular 5-in O.D. drillstring was used as hoist and bumpersubs providing 6.4 m total stroke allowed the regular HBM 1500 hammer to drive the pile without introducing shocks into the drillstring. Remote control of the piledriving operation proved to be an important feature. Adverse weather in the North Sea at this time of the year caused numerous interruptions to operations. The Sedco 445 is not capable of operating in waveheights in excess of 2.5 m and windforces over Beaufort 5-5. An HBM 1500 Hydroblok hammer was aboard and an identical hammer provided a backup. A standard self-supporting skid-mounted powerpack was also aboard. Two hydraulically powered reels completed the system. At the halfway point the heavy anchor chain was preconnected to the pile and some 60 ton of chain was lowered simultaneously with the hammer and the pile.

A complication was the requirement that the pile top had to be well below the mudline after driving. A short "follower" which remained connected to the hammer was re-used on all anchor piles. This solution proved to work well. Specially designed shearpins connected the anchor pile to the follower, allowing the combination of hammer, follower and anchor pile (including chain) to be lowered to the seafloor as a single unit. The shearpin connection was safely broken during the first hammer blows. Before the driving could begin two additional conditions were to be met:

- The connection of chain to pile required correct orientation; the Eastman Whipstock System solved this problem satisfactorily.
- Verticality of the pile had to be checked; the Regan Bubble System proved to be a good tool for the purpose.

On one pile the hammer had to be retrieved before final penetration was achieved. Relanding at a later stage was not a problem using standard equipment and the pile was easily installed. Total time for the first pile was approximately 100 hr, of which actual driving required less than 30 minutes. The last pile took less than 30 hr after crews became acquainted with the operation which had never before been attempted from a drillship. Actual driving times varied from 5 to 30 minutes, with the number of blows varying from 300 to 1800 at an average rate of 60 blows/min. Maximum bufferforce (impact-force) was 15 000 kN, maximum blowcount approximately 300. All driven piles have a final verticality of less than ±1 degree.

DRIVING PILES IN VERY DEEP WATER

A logical question that can be asked is whether the Puppet System could be used in "any depth"? After the experience in the 146 m North Sea depth and the prior experience in 320 m Gulf of Mexico the answer can be in the affirmative. In depths up to 400-500 m the presently available equipment and the systems that were used make us believe that such an operation does not pose new problems. For greater depths exceeding 500 m the energy supply in the present form with hoses will become prohibitive, if not from a technical point of view, economics will hardly be acceptable. An underwater powerpack, driven electrically, fed through an umbilical is a possibility that should be engineered, in case of sufficient interest. The Hydroblok hammer itself has all potentialities already at this stage to be submerged to very great depth. Its controllability remains the same.
Underwater piledriving experience is limited as yet; are its prospects promising? As far as the hammer is concerned, the Hydroblok hammer has proved to be an appropriate tool. Performance, reliability, remote controllability, all match the purpose. The underwater operations, done in various parts of the world, were very instructive.

One promising aspect in particular has strongly attracted our attention; it is the behaviour in water of a heavy mass such as a pile or a hammer, or the combination of both hanging from a cable or from a drillstring. We got used to rather uncontrolled, rather rapid movements of large masses in air. Once completely submerged, even influenced by currents up to 2 knots, the behaviour becomes surprisingly calm. Calm in the sense that the movement of the mass (say the piletip) is nearly nil, when the hoistable (crane boom) is kept in place and secondly a horizontal change of the crane boom position results in a very smooth and slow corresponding movement of the piletip; there are no wild movements at all.

The first few meters down from the watersurface in the situation at sea, however, is just the contrary; a mass hanging in this area is often brought in violent movement because of wave action. The forces resulting from such movements are notoriously high and for this reason one tends to think that such behaviour exists all the way down. This has clearly been proved not to be the case. Based on this knowledge, a number of "technical philosophies" were developed that can be fruitfully applied when designing handling systems for a particular offshore situation.

With respect to the watersurface area that the pile and hammer must pass, there are basically two principles. The one is, that the energy transferred from the wave action into the mass is accepted as well as the movement resulting therefrom. Such is the case when the mass can be lowered freely through this area without any lateral support, for instance with an offshore crane on sufficient boom-radius, where movements of the mass are acceptable, not causing damage. After being lowered deeper in the water, the mass is automatically calmed down. When working through the moonpool of a drillship, however, only limited movements of the mass can be accepted: a certain guide-system must be designed, that allows all the masses to be vertically transported in a guided sense until they have been lowered deep enough underwater to continue without guide. The guided portion can either be rigid or partly elastic allowing only a certain controlled lateral movement. For applications other than with a drillship a more universal system has been designed; the "Juggler"-system (Fig. 16).

Underwater Skirt Piles

Another problem is, how to land the pile into its pile sleeve, for instance in a cluster of pilesleeves, as is common for skirt piles of North Sea type platforms. Usually, as long as the driving takes place from above water, the pile and the followers are run down through pileguides connected around the platform leg at regular intervals. After the piles have been driven these pileguides have no function anymore and are often a nuisance.

It has been proposed to design a slim type underwater hammer, its length matching the pileguide spacing. On first sight this looks attractive, except for the handling of the hosebundle, which must pass through the guides as well, which certainly involves unacceptable risks. And there are more aspects that make such an approach less attractive and moreover in the end of such an operation the pileguides are still there, still being a nuisance.

The problem, however, can be looked upon in a different way, utilizing as much as possible the experiences of prior underwater piledriving and the ways their handling problems were satisfactorily solved. Within the scope of this contribution only two points will be described.
Based on well proven techniques, mainly underwater acoustics and underwater television (RCV), a pile can rather easily be lowered in a purely controlled sense. The last few meters are controlled by television. The principle here is not, as is common practice for other applications, to stab concentrically with the sleevepile. The lowering of the pile is done eccentrically, well away from the platform leg, until its tip has reached a level, only a few feet above the top of the sleevepile; this can be controlled by television (control in Z-direction). Than the pile is slowly moved in, thereby controlled in X and Y direction. For this purpose X, Y and Z markings are painted on the horseshoe type catch. The catch is mounted on top of the sleevepile and it is open to one side for easy reception of the piletip. The system is applicable for batter skirtpiles as well (Fig. 17).

The second point that I would like to raise is a question. The Cognac Platform (Fig. 5) is always referred to as a 3-part platform. Of course this is incorrect, because there are 3 parts. But shouldn't it be looked upon rather as a 2-part platform, built on a solid foundation? In conformity with sound engineering principles, as is common practice on land, the first thing one does, is to build a solid foundation, before the structure is erected. I believe that since the availability of a reliable underwater offshore hammer such sound engineering should become common practice in offshore engineering as well. Shouldn't we consider Fig. 5b as such a solid foundation for Cognac's 2-part platform? The question is, whether this is the most economical "solid foundation"? The author, not being a platform-designer by profession, may be excused for leaving this question unanswered here. Would further research in this direction perhaps lead to safer and cheaper subsea foundation techniques, where steel and concrete perhaps may prove to match well?

**EARLY PRODUCTION SYSTEMS**

Driving conductor pipes in early production systems can be an attractive technique. Conductor pipes with their usual top part configuration can easily be driven with a Hydrobloc hammer without being damaged; to prevent a maximum stresslevel being exceeded during driving the maximum allowable bufferforce can be calculated in advance. Accurate verticality is not a problem; all anchor piles in Piper Field (1978) were driven vertically with a tolerance less than ±1 degree. Driving times are short. A simple template, landed on the seafloor, can be leveled towards its first 3 or 4 cornerpiles, subsequently serving as a guide for all the other piles (or conductor pipes). There is no danger of any disturbance or washout of the soil. No grouting of the conductor into the soil is needed. Because the conductors are driven by a Hydrobloc hammer, the soilconsultant is given pertinent soil-information, derived directly from the driving, that enables him to judge the holding capacity of the conductor pipe in the soil. The driving can be done from a drilling vessel (Fig. 18). All piles and conductors being driven, the drilling operation can be started from the same vessel.

**PILED FOUNDATIONS FOR TENSION LEG PLATFORMS**

Tension Leg Platforms are being studied by many. All have in common that they require a very stable and reliable foundation in/on the seabed, generally in deep water, where inspection is hardly possible. Elaborate studies are well under way to gain knowledge of the behaviour of foundations under cyclic loading. The same applies to piled foundations. Other contributions will highlight these phenomena. They are beyond the scope of this contribution.
There are two contributions, however, that I would like to offer here in relation to TLP-foundations.

Important in this respect is to recall once more, that during Hydroblok-driving a great deal of (dynamic) soil information can be derived. Other recent publications have reported on this subject and there are more to follow, partly based or test-results as they were derived from recent HBM 4000 drivings on an 2.13 m pile on a testsite in the Netherlands.

The other contribution is a system to equally distribute the load over a number of piles (1, 2 or 3). Fig. 19 shows the principle of the PAP-system. The base frame primarily serves the purpose of stabilizing the anchor piles before they are driven. The anchor plate carries the anchorcable(s); the anchorcable-connection can be carefully checked and the cable is coiled before submersion. The anchor piles are driven, and a certain difference in the levels of the gimbals is acceptable. Pulling the anchorcable(s) makes the anchorplate match all gimbals, evenly distributing the anchorforce to all (maximum 3) piles.

The baseframe from this moment on has no function anymore, as far as vertical anchorforces are concerned. Pinning the baseframe firmly to the seafloor with separate piles makes horizontal components of the anchorforce not work any more on the vertical anchorpiles. Because of the nature in which TLP-anchorpiles are cyclically loaded, separation of horizontal and vertical force components can be advantageous.

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(5) HUSSEN, K. van "Submarine Piledriving for Deepwater Installations" Ocean Resources Engineering, September 1977, pp. 60-65

(6) JANSZ, J.W. "Subsea Piledriving: A Breakthrough" Petroleum Engineer, June 1978, pp. 76-86


(8) JANSZ, J.W. "The Puppet System"; A Simple Way to Drive Subsea Anchor Piles" Offshore Technology Conference 1979, OTC 3439
Table I: How variation of impact force with constant energy per blow affects penetration.

Hydrobloc Hammer, type HBM 3000.
Pile, 84-in O.D., 1.25-in. W.T.
Net energy per blow, constant 79 m. ton (produced by hammer).
Total driving resistance, constant 2100 ton.

<table>
<thead>
<tr>
<th>Impact force in t</th>
<th>1200</th>
<th>2100</th>
<th>3000</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portion of energy that pile is able to absorb: in m. ton (in % of net energy per blow)</td>
<td>65 (82%)</td>
<td>74 (94%)</td>
<td>54 (68%)</td>
</tr>
<tr>
<td>Penetration per blow in cm</td>
<td>0.1</td>
<td>2.2</td>
<td>1.4</td>
</tr>
<tr>
<td>Blowcount per ft</td>
<td>300</td>
<td>14</td>
<td>22</td>
</tr>
</tbody>
</table>

Schematic F-t diagrams

\[ F \text{ in tons}, \ t \text{ in milliseconds} \]

![Graphs showing F-t diagrams for impact forces of 1200, 2100, and 3000 tons.]
<table>
<thead>
<tr>
<th>Problem Area</th>
<th>Solution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Piledriving (6 500 ton bearing capacity)</td>
<td>Use hammer with sufficient capacity to be chosen from available types.</td>
</tr>
<tr>
<td>Under water piledriving (6 500 ton bearing capacity)</td>
<td>Use HBM 3000 A.</td>
</tr>
<tr>
<td>Lowering pile and hammer</td>
<td>Lower pile and hammer separately. (Not: pile + hammer; too heavy)</td>
</tr>
<tr>
<td>Lowering pile</td>
<td>Use elevator near top of pile.</td>
</tr>
<tr>
<td>Lowering hammer</td>
<td>Use elevator guidelines.</td>
</tr>
<tr>
<td>Landing hammer in desired position</td>
<td>Make guidelines end near top of pile.</td>
</tr>
<tr>
<td>Alignment of hammer on pile and lateral fixation</td>
<td>Use pilesleeve (Not: driving cage)</td>
</tr>
<tr>
<td>Retrieve hammer during/after driving</td>
<td>Use elevator guidelines</td>
</tr>
<tr>
<td>Allowance for barge-heave</td>
<td>Use yoke frame</td>
</tr>
<tr>
<td>Allowance for shock-wise movement of driving hammer</td>
<td>Use yoke frame (Not: bumpersubs)</td>
</tr>
<tr>
<td>Monitor during driving</td>
<td>Use all facilities for remote control that Hydroblok can provide. Redundant. (Not: divers)</td>
</tr>
<tr>
<td>Monitor during lowering hammer</td>
<td>Not required because of chosen elevator system. (Not: divers)</td>
</tr>
<tr>
<td>Monitor during lowering pile</td>
<td>Use RCV television and camera at piletip. (Not: divers)</td>
</tr>
</tbody>
</table>
Table III: Data for Puppet System method number 1 as used in Occidental's North Sea Piper Field.

Sea: waterdepth 146 m (480 ft)
current at surface of the water 1.2 m/s (4 ft/s)

Vessel: dynamically positioned with possible position changes 3 m (9.8 ft)

Soil: soft clay; pile penetration by selfweight of pile and hammer: 15 m (49 ft)

Pile: O.D. 1.5 m (60-in)
W.T. 2½-in
length 36.5 m (120 ft)
weight (above water) 84 ton (185 000 lbs)

Hammer: HBM 1500
weight (above water) 78 ton (172 000 lbs)
water displacement 29 m³
magnitude of maximum total soil resistance 27 000 kN (6 000 kips)

Table IV: Data for calculated example of Puppet System method number 2 using the Puppet Weight

Sea: waterdepth 146 m (480 ft)
current at surface of the water 1.2 m/s (4 ft/s)

Vessel: dynamically positioned with possible position changes 1.5 m (5 ft)

Soil: no pile penetration by selfweight of pile and hammer is assumed.

Pile: O.D. 1.5 m (60-in)
W.T. 2½-in
length 73 m (240 ft)
weight (above water) 168 ton (370 000 lbs)

Hammer: HBM 1500
weight (above water) 78 ton (172 000 lbs)
water displacement 28 m³
magnitude of maximum total soil resistance 27 000 kN (6 000 kips)

Puppet weight: steel, weight (above water) 85 ton (187 000 lbs)
Figure 1: Impact Force-Time diagrams.
Pile Test in Slievedrecht, the Netherlands October 1978
Hydrobloc HBM 4000; Pile 60 m, 84" O.D., 1.25" W.T.

* (No test results available for the exact calculated cases)
Figure 2: HBM 4000 type Hydroblok hammer, sitting freely on top of the 1:5 batter testpile while driving.

Figure 3: Schematic cross-section of the Hydroblok hammer showing the built-in buffer and the flat bottom anvil.
Figure 5a: Overall dimensions.

Figure 5b: The bottom section to be secured into the seabed.

Figure 5: Shell Oil's Cognac Platform in the Gulf of Mexico, composed of 3 parts.
Figure 6a: WRONG: hammer guidelines leading to base structure create undesired misalignment of hammer downward travel when landing on pile

Figure 6b: GOOD: elevator lines serving also as hammer guidelines results in small misalignment at pile top.

Figure 6: Design philosophy for elevator.
Figure 7: Elevator on 4 cables, suitable to carry the weight of the pile during its downward travel.

Figure 8: Picture of HBM 3000A on testpile with fully extended yoke frame.

NOTE: The loopwise hose connections between upper part of yoke frame and hammer are not yet connected here.
Figure 9a: Operations Analysis; Topview of hose drum assembly and hammer arrangement.
Figure 9b: Operations Analysis; Hammer on barge deck, hosedrum assembly away from side of the barge.

Figure 9c: Operations Analysis.
Hammer in line with operations ξ.
Hosedrum assembly ready to be shifted to barge side, allowing the hoses to hang vertically down.
Figure 11: The two basic Puppet System methods.

METHOD 1

- hoist (kept slack)
- puppet-weight-guidelines
- puppet-eyes
- hammer
- pile

Puppet Weight stabilizes pile and hammer independently of soil properties.

METHOD 2

Pile penetrates soil by selfweight and lateral soil resistance keeps pile and hammer upright.

Figure 10: Hofermuller's Barge US 16 rigged up with facilities to handle the piles and to drive them underwater.
Figure 12: Analysis of Puppet System method number 1; the graph shows pile angle $\alpha$ versus penetration depth during slackening of hoist for data according to table III.

Figure 13: Analysis of Puppet System method number 2; the graph shows moment $M_\alpha$ versus pile angle $\alpha$ for data according to table IV.
Figure 14: Capacities of Puppet System method number 2 for data according to table IV.

Figure 15: Realisation of a subsea anchorpile (North Sea 1978)
Figure 16: JUGGLER-system.
Universal system to pass waterline in a fully guided way.
Figure 17: Open catch type sleevepile entrance.
Figure 18: Underwater driving of conductor pipe in early production systems.
Figure 19: Piled Anchor Point (PAP) System with self-equalizing force distribution for Tension Leg Platforms.
A SIMPLE WAY TO DRIVE FREE-STANDING SUBSEA ANCHOR PILES

by J. W. Jansz and H. S. T. Brockhoff, Hollandsche Beton Groep N.V.

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This paper was presented at the 13th Annual OTC in Houston, Tex., April 30-May 3, 1979. The material is subject to correction by the author. Permission to copy is restricted to an abstract of not more than 300 words.

ABSTRACT

The paper describes the "Puppet System" that have made it possible to drive unsupported piles in deep water without the assistance of any structure on the sea floor to provide lateral stability. Theory as well as recent North Sea experience is presented. The system is ideally suitable for subsea anchor piles.

A "Puppet System" makes use of an underwater Hydroblok hammer. The system allows the largest Hydroblok hammer to be used in combination with a very large pile. The hammer carries the pile during its downward travel from the surface vessel. It is a temporary connection that keeps pile and hammer safely together until the driving is started.

Correct positioning, and giving the pile a certain orientation prior to driving, as well as bringing the pile into the required verticality or batter and maintaining it in that position have been exercised; monitoring the operation showed that all functions could be kept under full control using existing remotely controlled devices.

Mathematically the problem has been fully assessed and a computer program has been made, that takes into account all possible influences.

"Puppet Systems" in principle can be used in any waterdepth; they do not require diver assistance. Anchorpiles in deeper water may serve TLP foundation purposes as well as OTEC's. The paper shows an example that has been worked out for a waterdepth of 1500 m.

Utilization of the Hydroblok hammer in the "Puppet System" yields real soil information, because of the hammer's ability to derive such dynamic soil properties while actually driving.

Late 1978 the system found practical application for the first time, when a number of 60-in O.D. piles were driven in 480 ft waterdepth in the North Sea, without any lateral support other than the interaction between the soil and the pile. The driven piles serve as anchor piles, their verticality is well within ± 1 degree.

This operation involved two extra complications. A pre-attached heavy anchor chain connected approximately halfway the pile had to go down simultaneously with the hammer and the pile. The top of the pile had to be driven to a certain distance below the sea floor.

SUBSEA ANCHOR PILES DRIVEN UNSUPPORTEDLY

All offshore piledriving projects, as far as our knowledge goes, have one aspect in common; there is always a structure of some kind sitting on the sea floor prior to driving. The structure generally has sleeves to receive the pile. Once stabbed the pile derives its stability from the structure. Driving piles into Shell Oil's Cognac platform (1977) in the Gulf of Mexico, amongst other projects, was such an example.

But, is it possible at all to drive a free standing pile underwater without a structure? It has been a desire for many years to be able to drive heavy anchor piles into the seabed without the help of a template to keep the pile upright at the moment of touch-down of the pile-toe and during the first hammer blows.

After two years of research the answer could be given. It has been named the "Puppet System". There are two methods and both will be described in this contribution; not only the underlying theory, but also the first practical application in the North Sea will be reported in the following chapters.

References and illustrations at end of paper
Before doing so, however, we like to ask your attention for a short summary of features that ultimately have made it possible to arrive at the "Puppet Systems" of today. There is logic in this step by step development, and it may be expected that corresponding logic may as well stretch further into the future. In our opinion not any one of these features could have been omitted.

The Hydroblok piledriving hammer plays an important role in this respect (references 1, 2, 3, 4 and 5). It had first of all to be proved in practice that there was a piece of equipment available to suit the purpose of subsea anchor piledriving. What are those features?

1. **The built-in buffer (figure 1).** Now an all-steel anvil can be introduced that transfers energy and force in a steel-to-steel manner into the pile, not requiring any capfilling material. Apart from considerable energy losses in such material which have a negative effect on performance, its replacement at regular intervals prevents such a hammer from being used as an underwater tool.

2. **Hammer protection.** Specifically for the larger hammers it becomes essentially important to protect the hammer from excessive stress and strain. At moment of impact shock-waves propagate not only in the pile, but also in the hammer. The buffer controls the peakforces and consequently they can be kept within acceptable limits. Now it has become possible to design the various hammer parts properly on fatigue. For the Hydroblok hammers extensive stress and strain calculations have been made, both statically and dynamically, the latter being specifically based on known impact speeds of the various colliding hammer parts. Only after this research stage, when extensive test measurements during real piledriving have proved that the complex calculations correctly assess what happens in practice, it may be expected that the hammer is designed as a reliable piece of equipment, suitable to be used underwater during longer periods.

3. **Remote Controllability.** The built-in buffer allows the impact force to be constantly indicated or recorded. Kinetic energy of each blow near moment of impact is also automatically measured and displayed or recorded. Any type of operation that takes place out of sight or sound can only be mastered when certain key values that change during the operation are numerically known. And this also applies to driving piles underwater.

4. **The Pile Sleeves, located in the lower part of the hammer (see figures 1 and 12).** The flat-bottom anvil is captively held in the top of the pile sleeve. The lower part of the pile sleeve generally serves as an open conical guide-bell, which transits into a cylindrical part that serves the purpose to guide onto the pile. In this way the Hydroblok hammer can be seated on top of the pile, thereby securing a perfect square in-line transmission of the impact force from the ram into the pile. Also raked piles with a batter of 1:5 that have sufficient stiffness of themselves can be driven in this way; the hammer does not require any additional support nor guide.

5. **Hammer Safety Devices.** Once seated on the pile, the hammer can be started or it can be set to start automatically, provided everything is in the "go" order. Built-in safety devices automatically stop the ram movement immediately at the moment that a "no-go" situation arises which could damage the pile-hammer system, as for instance occurs when a pile suddenly starts to run. As soon, however, as the "go" situation restores, the driving of the hammer restarts automatically or can be restarted manually if preferred.

6. **Lowering Pile and Hammer as One Unit.** The pile sleeve has also provided the possibility to lower the pile with the hammer on top as one unit in one single operation. Certain provisions prevent the hammer from loosing the pile during their mutual descent. Shortly after the driving has started, the temporary connection between hammer and the pile is disconnected allowing the hammer to be retrieved.

These provisions have been used for instance in the 1976 North Sea subsea anchor pile project.

Not before all these aspects are solved satisfactorily, one can think of driving piles in greater water depths (reference 6). The practicability and also the reliability of all these features were clearly proved during 1977, when apart from some other subsea piledriving projects of more common dimensions, it was Shell Oil's Cognac platform where the 24 giant 622 ft long piles in one end were driven 492 ft deep into the seabed in over 1000 ft water depth (references 7 and 8).

Now in 1979 it may be stated that there is a reliable underwater hammer available. And in 1979 the next logical step in the underwater piledriving development are the "Puppet Systems".

**SIMPLE METHODS; THE PUPPET SYSTEMS**

When a pile hangs freely only supported from its top, it is in positive balance. This is the case in air as well as underwater, where outside forces such as wind or currents may affect this balance, but nevertheless we call it a stable balance, because it is a self-stabilizing system. This is correct, as long as the pile toe is not supported, for instance by the sea floor. In that case the pile system becomes unstable, especially so when there is a hard soil; the pile will topple.

To solve the problem it soon became clear that a stabilizing force must be introduced, preferably one that acts automatically and that comes into force at the right moment and in the right sense.

And the stabilizing system should also be as simple as possible, preferably utilizing known and well-proven techniques only.

And last but not least the system should not unnecessarily complicate the total operation.
And the system should be extendable to greater depths, say 1500 to 2 000 m (6 000 ft).

In the research period (2 years) a variety of possible methods were studied, of which finally two were selected for application in practice (figure 2):

- Puppet System number 1; a method where only lateral soil is utilised to stabilize pile and hammer.

- Puppet System number 2; a solid mass, the Puppet Weight, provides self-stabilization to both pile and hammer; this method works independently of soil properties.

Both methods are realistic, because of the features mentioned earlier in this contribution, that are available in the Hydroblock underwater piling driving hammers (figures 1 and 12). This hammer is an integral part of the Puppet System.

In a Puppet System operation the pile and the hammer are temporarily connected for easy lowering of both as one single unit. Several proven connector systems are available for this purpose; all utilise the Hydrobloc block hammer's pile sleeve (figures 1 and 12). One system shows a ring welded near the top of the pile, that is gripped by a hydraulically operated device inside the pile sleeve. A second system is based on specially designed fail-safe shear-off connectors, which are automatically sheared-off during the first hammer blows; this system proved its reliability during the Puppet System operation in the North Sea Piper Field (figure 14).

Both types of temporary pile-hammer connectors have in common that they allow the Puppet System to be used without any diver assistance. The most simple solution is to lock the pile to the hammer's pile sleeve with short chains. This method, however, requires a diver to disconnect the chains, prior to hammer retrieval.

WORKING PRINCIPLE OF PUPPET SYSTEM NUMBER 1

Seabeds with thick layers of mud or soft soil may show sufficient lateral supporting capacity to stabilize the pile with the hammer on top, because of sufficient self-penetration of the pile. Apart from soil properties, it also depends on the pile's deviation from the vertical (pile angle θ) whether lateral soil resistance will keep the pile with the hammer upright. Figure 6 shows for various penetration depths the pile rotation resistance moment M₀versus an increase of pile angle θ as was to be expected in the North Sea Piper Field project, where soil properties according to table I were available. Figure 6 (thin line) shows also the overturning moment M₀of the free-standing pile and hammer (hoist slack) based on the assumption that the pile has been penetrated fully in an exactly vertical position (it should be recon sidered that the horizontal axis of figure 6 represents an increase of pile angle θ).

Comparison of this line with the M₀-line for full penetration (z = 15 m) shows that the pile rotation resistance moment M₀ far exceeds the overturning moment M₀. Thus, it may be expected that the soil provides sufficient stability to keep the pile and hammer upright (this Piper Field example will be analysed further hereafter, taking into account currents and a possible penetration of the pile with a certain deviation from the vertical). Because the deviation of the pile varies with increasing penetration, the full self-penetration process must be studied in order to assess the correct pile angle θ at moment that the soil starts to bear the full weight of pile and hammer. Assuming a constant distance q between the upper side of the hoist and the pile toe (equal to an unchanged position of the vessel), the variation of pile angle θ versus the increasing pile penetration z can be established by solving the differential equation which describes the mechanical behaviour of the system (for derivation see figure 3 and appendix A).

\[
\frac{da}{dz} = \frac{1}{3M_0} \left[ \frac{q(2-q(h_z)^2)}{(1-q(h_z)^2)^2} \right]
\]

Both vertical soil resistance R and pile rotation resistance moment M₀ are soil properties, whereby R is assumed to be a function of pile penetration z only and M₀ is a function of both z and pile angle θ, thus \( \frac{dR}{dz} \) and \( \frac{dM_0}{d\theta} \) can be derived.

With known soil properties and a given initial pile angle θ at the beginning of the penetration, the differential equation (1) can be solved for known values of the parameters q, h₁, l₁, l₂, G₁ and G₂ (all assumed to have a constant value during self-penetration of the pile). A computer program has been developed to assess the right values for pile angle θ versus penetration depth z.

OPERATION OF PUPPET SYSTEM NUMBER 1

As indicated before, a Puppet System operation mainly starts with the temporarily connection of pile and hammer into one unit. There must be some sort of a hoist (e.g. drillstring) to lower the unit. Figure 4, stage 1, shows the pile shortly before it will touch the sea floor. Because of current-action the pile toe will not remain vertically beneath the vessel, but will deviate a certain distance q₁ (landing distance):

\[
q_1 = F_p \left( \frac{h_1}{G_1 + G_2} + \frac{l_1 (l_1 - l_p)}{G_1 (l_1 - l_1) + G_2 (l_1 - l_2)} \right)
\]

F_p represents the resultant of all current forces on the system acting at a distance l₀ above the pile toe. Further symbols will be clear from figures 3 and 4. The corresponding pile angle q₁ (landing angle):

\[
q_1 = \frac{F_p (l_1 - l_p)}{G_1 (l_1 - l_1) + G_2 (l_1 - l_2)}
\]
To make the calculations to be conservative, a possible position change \( \Delta q \) of the vessel will be assumed to occur in the time-split between touch-down and the beginning of the pile's self-penetration (stage 3). This results in a horizontal distance \( q_3 \) (stabbing distance) between pile toe and vessel, with corresponding pile angle \( \alpha_3 \) (stabbing angle):

\[
q_3 = q_1 + \Delta q
\]

\[
\alpha_3 = \frac{5a_1(G_1+G_2) - F_p 1_h (h - 1_a)}{h.1_a (G_1+G_2) - (G_1G_2) (h-1_h)}
\]  

The position change \( \Delta q \) can be in all possible directions. However, for this Puppet System Number 2 an upstream position change (\( \Delta q \) positive) will always have a more negative effect than a downstream one.

The pile penetrating into the sea floor (stage 3) will consequently lead to a decrease of the hoist force \( S \), ultimately ending up in complete bearing of the pile and the hammer in the seabed. If necessary, the vessel may change its position during the pile-penetration, to obtain a verticality of the pile within the required tolerances, or even a required batter position of the pile. The pile's deviation from the vertical can simply be measured by using the Regan Bubble System, which has been proved its reliability in this specific application during the Puppet System piling driving operation in the Piper Field (Figure 15). Once the pile has fully self-penetrated the hammer will be started to operate; the hoist being kept slack (stage 4).

**ANALYSIS FOR OCCIDENTAL'S NORTH SEA PIPER FIELD**

The theory above will be illustrated here with the Hydrobiol hammer IBM 1500 driving 1.5 m (60-in) O.D. by 36.5 m (120 ft) long anchor piles into the Piper Field. Soil properties according to table I are available; further data are specified in table II. As mentioned earlier, vertical soil resistance \( R \) as a function of \( z \) and pile rotation resistance moment \( M_z \) as a function of \( z \) and \( \alpha \) must be given. In this soft clay, the vertical soil resistance \( R \) versus \( z \) will be assumed according to figure 5, taking into account the given soil condition that this pile and hammer will penetrate 15 m by self-weight only (table I). In mathematical form:

\[
R = Cz^2, \text{ hence } \frac{dR}{dz} = 2Cz
\]

where \( C \) denotes a certain constant as derived in figure 5.

For soft clay \( M_z \) as a function of \( z \) and \( \alpha \) can be derived with use of API RP 2A (reference 14). This derivation is not shown here, because it is irrelevant for this paper. Figure 6 shows the result (full lines). Differential equation (1) can only be solved easily by numerical method when a linear relationship of \( M_z \) versus \( \alpha \) is presumed. A conservative linearized relationship according to the dotted lines in figure 6 is assumed; this relationship will be checked afterwards to make sure that it was not too optimistic.

Substitution of equation (6) in equation (4) leads to the differential equation:

\[
\frac{d\alpha}{dz} = \frac{2C.1_h (\frac{\alpha - q/h}{1 - 1_a/1_h}) \frac{z}{h}}{k_s + (G_1+G_2-C_z^2)[\frac{1}{1 - 1_a/1_h} - (G_1G_2)]}
\]  

where \( k_s \) denotes the relationship \( \frac{dM_z}{d\alpha} \) according to the dotted lines in figure 6 (\( \frac{dM_z}{d\alpha} \) is a function of \( z \) only).

The horizontal distance between the vessel and the pile toe just before touch-down follows from equation (2):

\[
q_1 = 1.5 \, m
\]

This distance and an upstream position change (worst case) of 3 m (table II) applied to equation (4) and (5):

\[
q_3 = 4.5 \, m
\]

\[
\alpha_3 = 2.5 \, degrees
\]

In a conservative calculation, where no re-positioning of the vessel during the pile's self-penetration is assumed (figure 4, stage 4), the initial pile angle \( \alpha \) at the beginning of the self-penetration is equal to the pile stabbing angle \( \alpha_3 \). With this initial pile angle of 2.5 degrees and the given data, differential equation (7) can be solved by numerical mathematical method (a computer program is available). This will render as a result pile angle \( \alpha \) versus penetration depth \( z \). From equilibrium of forces it follows that \( S = G_1+G_2- R \). Substitution of \( R \) according to equation (6) will render \( S \) as a function of \( z \). Hence, the required pile angle \( \alpha \) versus the decreasing hoist force \( S \) is known. Figure 7 shows the result. The total increase of \( \Delta \) during the pile's self-penetration, thus the decrease of hoist force \( S \) appears to be approximately 3 degrees. This value compared with figure 6 illustrates that the linear relationship (dotted lines) indeed is more pessimistic than the known relationship (full lines), so the calculation really is conservative. Therefore, it can be expected that in reality the deviation of the pile from the vertical will be less than 4 degrees (figure 7, hoist fully slackened off).

The Piper Field results, where driven piles have a final verticality with a tolerance of less than \( \pm 1 \) degree, show that this analysis is really very conservative; it should only be used to judge whether Puppet System number 1 can safely be used or that number 2 is necessary.

**WORKING PRINCIPLE OF PUPPET SYSTEM NUMBER 2**

The soil-independent Puppet System number 2 (figure 2) requires two puppet-weight-guidelines, running down from the vessel, passing through the puppet-eyes, to the Puppet Weight.
The latter is simply a mass, loosely slipped around the pile at a low level, thus producing a tension force in the puppet-weight-guidelines. Component of this tension force acts onto the puppet-eyes, thus stabilizing pile and hammer within certain limits which will be explained hereafter.

Figure 8 shows how the mechanical system works. For simplicity sake in first instance the following is assumed: (1) no friction between the puppet-weight-guidelines and the puppet-eyes, nor between the Puppet Weight and the pile; hence, the hoist force $T$ is equal to $G_3 \cos \alpha$; (2) the vessel remains exactly vertically above the vessel (no position change of the vessel); (3) no currents, thus $F_P = 0$; (4) the Puppet Weight remains at a constant level above the sea floor, thus distance $l_3$ is constant and independently of pile angle $\alpha$. Suppose the system to be out of balance; let $M_3$ be the moment of forces directed towards a positive value of $\alpha$, relative to the toe side of the pile. From statics it follows (figure 8):

$$ M_3 = (G_{11} + G_{12} + G_{13}) \sin \alpha + $$
$$ -G_{12} \sin(\alpha - \beta) (\cos \alpha) $$

where $\beta = \arctan\left(\frac{-l_3 \sin \alpha}{h - l_4 \cos \alpha}\right)$.

For a sufficient large value of $G_3$ (which means a sufficient heavy Puppet Weight), this curve is similar to the curves shown in figure 10 (however, symmetrical with respect to the vertical axis). The system is in equilibrium in the positions EP and UEP, where $M_3 = 0$. Position EP is the only stable Equilibrium Position; where in this simplification $\alpha = 0$. Positions UEP are Unstable Equilibrium Positions. The only relevant question in this stage is whether the stable Equilibrium Position EP exists or not. From mechanics (reference 10) it follows that the system remains stable as long as $dM_3/d\alpha < 0$, where $M_3$ is the moment of forces directed towards a positive value of $\alpha$.

Derivation and linearization (small values of $\alpha$) of equation (8) gives for the equilibrium position where $M_3 = 0$ and $\alpha = 0$:

$$ \frac{dM_3}{d\alpha} = (G_{11} + G_{12} + G_{13}) \alpha - \frac{G_{12} l_2 h}{h - l_4} $$

(9)

Hence, the equilibrium position where $\alpha = 0$ is stable if:

$$ G_3 > \frac{(G_{11} + G_{12})(1 - l_4/h)}{l_2 - l_3(1 - l_2/h)} $$

(10)

This simple equation specifies the minimum required Puppet Weight to make the system self-stabilizing. Because of the simplifications mentioned earlier in this paragraph, this equation may only be used to assess an approximation of the required Puppet Weight; a full analysis taking into account all mentioned aspects will answer the question whether a certain Puppet Weight stabilizes the system properly.

In appendix B the moment of forces $M_3$ has been derived, taking into account friction forces in the system, position changes of the vessel, currents and changes of $l_3$ due to a pile angle $\alpha$. Figure 10 shows the moment of forces $M_3$ for data from table III. When friction forces in the system are taken into account there are two curves; one because of a downward movement of the vessel and a second one for an upward movement. In this graph the points EP, UEP, RS and URS are the important ones.

Point EP is the only (stable) Equilibrium Position of the system, because of $dM_3/d\alpha < 0$. A small deviation of the system away from this position will always cause a moment $M_3$ that will direct the system back towards position EP.

Points UEP are Unstable Equilibrium Positions of the system; these points have no practical value. Points RS are to be looked upon as Re-Stab positions. If for some reason pile angle $\alpha$ would reach one of these positions, the system will not regain automatically its (stable) equilibrium position EP when the vessel moves upwards at that same moment; the pile must be lifted and re-stabbed again. Does the vessel show a downward movement at that particular moment, re-stabbing could theoretically be put off until the system reaches point URS, being an Utmost Re-Stab position. In practice, however, only the positions EP and RS are of interest. The other points (UEP and URS) therefore will not be considered any further in this paper.

A computer program is available to assess the points EP, RS and URS, however these points can also be assessed easily by plotting the moment of forces $M_3$ versus pile angle $\alpha$.

**OPERATION OF PUPPET SYSTEM NUMBER 2**

The first two stages of a Puppet System number 2 operation (figure 9) are identical to those for number 1 (figure 4). Equations (2), (3) and (5) will change slightly due to the Puppet Weight $G_3$:

$$ q_1 = \frac{F_P(h - l_3)}{G_1 + G_2 + G_3} + \frac{l_3(l_3 - l_p)}{G_1(l_4 - l_3) + G_2(l_2 - l_3) + G_3(l_3 - l_1)} $$

(11)

$$ q_3 = \frac{q_1 + \Delta q}{G_3} $$

(12)

$$ q_3 = q_1 + \Delta q $$

(13)

$$ q_3 = \frac{G_3 l_2 (G_1 + G_2 + G_3) - F_P l_p (h - l_3)}{h l_4 (G_1 + G_2 + G_3) - (G_1 l_1 + G_2 l_2 + G_3 l_3) (h - l_4)} $$

(14)

Again, the position change $\Delta q$ can occur in all possible directions; it depends on the numerical value of current velocity and position change which position change represents the worst case.
Stage 3 (figure 9) shows the pile after being stabbed and the hoist is fully slackened off. The tension forces in the puppet-weight-guidelines now stabilize the system in such a way that the pile will be kept in its (stable) equilibrium position with a slight deviation from the vertical because of currents and distance q. The system may now undergo various position changes $\Delta q$, each time leading to a new equilibrium position EP.

Before pile driving starts the vessel may change its position to obtain a vertical position of the pile (stage 4) or if required a batter one. When the pile's position is within the required limits, the hammer will be started to operate while the Puppet Weight keeps pile and hammer upright. No dangerous shocks can be transferred into the hoist, nor in the puppet-weight-guidelines because of the slack in the hoist and the loosely guidance of the Puppet Weight around the pile. After some hammer blows the pile has penetrated the soil far enough to remain in its upright position, even without the Puppet Weight.

PRACTICAL DEEP WATER EXAMPLE

How does the above outlined theory works out in deep water?

It is assumed here that 60-in O.D. heavy anchor piles have to be driven in 1500 m water depth to serve as moorings (reference 11) for an OTEC plant (Ocean Thermal Energy Conversion). The operation will be performed from a dynamically positioned vessel with an accuracy of its position within 1 percent of the water depth. Puppet System number 2 will be used to stabilize the hammer; in this example an HBM 1500 type. Current near the sea floor is of importance. Here it is assumed that when pile and hammer are nearing the sea floor, both will experience currents of 0.3 m/s; this is expected to be a safe criterion for most potential OTEC sites (references 12 and 13).

The relevant data for this operation are specified in table III. Both an upstream position change of 1% of the water depth and an equal downstream one are assumed. Without taking into account current, position changes and friction into the system a Puppet Weight heavier than 119 ton (above water) would be required (following from equation (10)). From some analyses with various Puppet Weights it was found that a 140 ton heavy Puppet Weight would stabilize the system properly.

For this Puppet Weight table IV shows the relevant results analyzed according to the theory above. An important parameter to judge the system's capacity is the smallest angle between the equilibrium position EP and any re-stab position RS. The downstream position change appears to be the worst in this respect. Figure 10 shows the moment $M_h$ versus pile angle $\alpha$ for this case with the important points EP, RS and URS, as discussed before.

Figure 11 finally shows these calculated results in a combined picture, where pile angles have been drawn 2 times enlarged. In this example where a 140 ton (310 000 lbs) Puppet Weight stabilizes a pile of equal weight and the pile driving hammer, the deviation of the pile from the vertical will be kept between +4.2 degrees and -5.0 degrees; the "pile movements area". The system is fully self-stabilizing, as long as the pile is in any position between +8.6 degrees and -8.5 degrees; "the self-stabilizing working area". Exceeding the ultimate pile angles of this area for any one reason, means that the pile and hammer could remain in the slanted position, safely supported by the hoist. The operation must be repeated and the pile must be re-stabbed ("re-stab areas").

The pile stabbing angles follow from equations (11), (13) and (14): $\Delta \alpha = +1.2$ degrees for an upstream position change and $\Delta \alpha = -1.0$ degree for a downstream one. These angles provide the "pile stabbing area", well within the self-stabilizing working area (figure 11). It says that during pile stabbing the system will always automatically remain within the pile movements area.

TODAYS EXPERIENCE AND WHAT IS ABOUT TO COME

Todays experiences with respect to offshore underwater pile driving, though still moderate in number, are promising. Shell Oil's Cognac underwater pile driving in 1977 was a great success (reference 7 and 8). So were a few more subsequent subsea pile driving operations, all of a considerable less sophisticated nature. But all had in common, that the Hydrolok hammer performed good.

The latest experience at the time this paper was composed is the first application of the Puppet System. Some scattered information on this project has been published (reference 9). We will shortly summarise what the operation really looked like.

Late 1978 (August, September) eight anchor piles were driven into the seabed in Occidental's North Sea Piper Field. The nature of the soil at this location was as given in table I. The project was the first of this type and was conducted from the Sedco 445 drillship which is dynamically positioned. Only minor adoptions to the ship's installations were required. Regular 5-in O.D. drillstring was used as hoist and bumpersubs providing 21 ft total stroke allowed the regular HBM 1500 hammer to drive the pile without introducing shocks into the drillstring. Remote control of the pile driving operation proved to be an important feature. Adverse weather in the North Sea at this time of the year caused numerous interruptions to operations. The Sedco 445 is not capable of operating in waveheights in excess of eight ft and windforces over Beaufort 5-6. An HBM 1500 Hydrolok hammer (figure 12) was aboard and an identical hammer provided a backup. A standard self-supporting skid-mounted powerpack was also aboard. Two hydraulically powered reels completed the system.
At the halfway point the heavy anchor chain (figure 13) was reconnected to the pile and some 60 ton of chain was lowered simultaneously with the hammer and the pile. A complication was the requirement that the pile top had to be well below the mudline after driving. A short "follower" which remained connected to the hammer was re-used on all anchor piles. This solution provided to work well. Specially designed shearpins connected the anchor pile to the follower (figure 14), allowing the combination of hammer, follower and anchor pile (including chain) to be lowered to the sea floor as a single unit.

The shearpin connection was safely broken during the first hammer blows. Before the driving could begin two additional conditions were to be met:

The connection of chain to pile required correct orientation; using the Eastman Whisp Stop System solved this problem satisfactorily.

Verticality of the pile had to be checked; the Regan Bubble System (figure 15) proved to be a good tool for the purpose.

On one pile the hammer had to be retrieved before final penetration was achieved. Relanding at a later stage was not a problem using standard equipment and the pile was easily installed. Total time for the first pile was approximately 100 hr, of which actual driving required less than 30 minutes. The last pile took less than 30 hr after crews became acquainted with the operation which had never before been attempted from a drillship. Actual driving times varied from 5 to 30 minutes, with the number of blows varying from 300 to 1800 at an average rate of 60 blows/min. Maximum bufferforce (impact-force) was 15 000 kN (3300 kips), maximum blowcount approximately 300. All driven piles have a final verticality with a deviation less than ±1 degree.

Concerning the near future there are two directions that may be discerned. The one points to increasing depth and the other points to adapted constructions, since a reliable underwater hammer has become available.

With respect to greater depth it is expected that the Puppet System can be used in its present form. The above example for 1500 meter confirms its feasibility. Concerning the underwater hammer energised from the surface it is expected to serve depths up till some 400 - 500 meter.

In greater depths the energy supply in the present form with hoses will become prohibitive; if not technically, economics will become unacceptable. An underwater powerpack driven electrically, fed and controlled through an umbilical from the surface is the logical next step. The system must be engineered in due course, when sufficient interest arises. The Hydrobloc hammer itself has all potentialities already in this stage to be submerged to very great depth; its controllability remains the same.

Concerning structures that rest on the sea floor, whether they reach to the surface or not, the situation may change in due time, when gained experiences with underwater piledriving will become more publicly known in their various aspects.

Only a few examples can be given here. Such as for instance the behaviour of a pile when lowered in the sea; its movements are so much different as compared to the same pile moving in air or the first tens of feet down from the water surface. The behaviour of a heavy body, hanging from a surface vessel deep below the surface makes other handling procedures come into the picture, including stabbing, orienting and positioning. Monitoring at greater depth with closed television circuits is as if one's eye really was on the spot. Combination of these techniques, partly already existing and in use for many years, partly only applied yesterday, will render new installation techniques and consequently new designs of offshore structures. Well known techniques such as the Eastman Whisp Stop System and the Regan Bubble System (figure 15), regular 5° drillstring, universal joints and bumber subs used in combination with the Hydrobloc hammer, its specific features and the Puppet System has resulted in dependable subsea anchor piles. The method can be improved in various aspects, but 1978 Piper Field rendered evident proof of its viability.

So other applications possibly may come within reach. For instance for early production systems, where driven conductor pipes can become an attractive technique. Conductor pipes with their usual top part configuration can easily be driven with a Hydrobloc hammer without being damaged; to prevent a maximum stresslevel being exceeded during driving the maximum allowable bufferforce can be calculated in advance. Accurate verticality is not a problem; all anchor piles in Piper Field were driven vertically with a deviation less than ±1 degree. Driving times are short. A simple template, landed on the seafloor, can be leveled towards its first 3 or 4 cornerpiles, subsequently serving as a guide for all the other piles (= conductor pipes). There is no danger for any disturbance or washout of the soil. No grouting of the conductor into the soil is needed. Because the conductors are driven by a Hydrobloc hammer, the soil consultant is given pertinent soil-information, derived directly from the driving, that enables him to judge the holding capacity of the conductor pipe in the soil. The driving can be done from a drilling vessel (figure 16). All piles and conductors being driven, the drilling operation can be started from the same vessel.

Dependable foundations for Tension Leg Platforms is another subject, where underwater driven piles may be the solution. Here the property of a Hydrobloc hammer to render dynamic soil information while driving, may prove to be of indispensable value, because such piles are subject to cyclic loadings. It is noteworthy in this respect to know that shortly before this paper was given shape the testing was finished on a test pile 84-in. O.D., 1.25-in. wall thickness) that was driven by an HDM 4000 type Hydrobloc hammer. As far as the measured signals could be judged in first instance they look consistent. Their analysis, however, will take time. The test is partly sponsored by a number of oil companies and certifying organisations, who will receive the final report.
Main subject of this test, as far as the Hydrobloc organisation is concerned is to develop a measuring technique, based on the Hydrobloc hammer Force-Time history related features, that must clearly unveil dynamic friction separate from tip resistance continuously during driving not requiring sensors on the pile, except near the top of the pile. Where the top of the pile is in the pile sleeve this is the only acceptable place where such sensors are acceptable.

It is believed that such a solution would contribute to the applied science of driving piles in general, and specifically so in subsea piledriving for near future next generation platforms and offshore structures.

\[ \alpha = \text{pile angle measured from the vertical (positive value according to figure 3 and 8)} \]
\[ \alpha_1 = \alpha \text{ at pile landing (just before touch-down)} \]
\[ \alpha_s = \alpha \text{ just before pile stabbing (figure 4 and 9, stage 2)} \]
\[ \beta = \text{angle of hoist (figure 3) or puppet-weight-guidelines (figure 8), measured from the vertical (positive value according to figure 3 and 8)} \]

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APPENDIX A

Differential Equation for Puppet System Number 1

See figure 3. Reaction force $R$ and pile rotation resistance moment $M_p$ are assumed to act on the pile toe. From equilibrium of forces:

$$ S = G_1 + G_2 - R \quad \text{(A1)} $$

Moment of forces with respect to pile toe:

$$ M_x = (G_1 L_1 - G_2 L_2) \alpha - F_p L_p - M_p + S l_0 (\alpha - \beta) \quad \text{(A2)} $$

From geometry ($h =$ water depth, $q$ see figure 4, stage 3):

$$ (\alpha - \beta) = \frac{\alpha - q/h}{1 - l_0/h} \quad \text{(A3)} $$

Substitution of (A1) and (A3) in equation (A2):

$$ M_x = (G_1 L_1 + G_2 L_2) \alpha - F_p L_p - M_p + $$$$ - (G_1 + G_2 - R) l_0 \frac{(\alpha - q/h)}{1 - l_0/h} \quad \text{(A4)} $$

During the pile's self-penetration the system remains in equilibrium, hence $dM_x = 0$.

$M_p$ is a function of $\alpha$ and $z$, hence:

$$ dM_p = \frac{3M_p}{\partial \alpha} d\alpha + \frac{3M_p}{\partial z} dz \quad \text{(A5)} $$

This applied to equation (A4) (it should be reminded that $R$ is a function of $z$ only):

$$ dM_x = (G_1 L_1 + G_2 L_2) d\alpha - \frac{3M_p}{\partial \alpha} d\alpha - \frac{3M_p}{\partial z} dz + $$$$ - (G_1 + G_2 - R) l_0 \frac{1}{1 - l_0/h} d\alpha + $$$$ + l_0 \frac{(q - q/h)}{1 - l_0/h} \frac{dR}{dz} dz \quad \text{(A6)} $$

In general, when a pile penetrates soil with a constant deviation from the vertical, no pile rotation resistance moment will be generated; hence:

$$ \frac{3M_p}{\partial z} dz = 0 \quad \text{(A7)} $$

This applied to equation (A6) and written in a different form leads to:

$$ \frac{d\alpha}{dz} = \frac{l_0 \frac{(q - q/h)}{1 - l_0/h} \frac{dR}{dz} - \frac{M_p}{\partial \alpha} + (G_1 + G_2 - R) \frac{l_0 (q - q/h)}{1 - l_0/h} - (G_1 L_1 + G_2 L_2)} }{\frac{3M_p}{\partial z}} \quad \text{(A8)} $$

APPENDIX B

Moment of Forces for Puppet System Number 2

See figure 8. Let $M_2$ be the moment of forces directed towards a positive value of $\alpha$ and relative to the pile toe. From statics it follows:

$$ M_2 = (G_1 L_1 + G_2 L_2 + G_3 (L_3 + \Delta L_3)) \sin \alpha + $$$$$ - F_p L_p - T_1 l_0 \sin(\alpha - \beta) \quad \text{(B1)} $$

where: $\Delta L_3 = L_3 - h + h - l_3 \cos \alpha \cos \beta$

and: $\beta = \arctan \frac{q - l_3 \sin \alpha}{h - l_3 \cos \alpha}$

From figure 8A:

$$ W_1 = \pm 2 f_1 T \sin \alpha |\alpha| \quad \text{(B2)} $$

$$ W_3 = \pm f_3 G_3 \sin \alpha |\alpha| \quad \text{(B3)} $$

The $\pm$ sign of $W_1$ and $W_3$ depends on the vertical movement of the vessel. An upward movement will lead to an upward movement of the puppet-weight-guideline: positive sign. The negative sign goes with a downward movement.

From equilibrium of forces in the puppet-weight-guidelines:

$$ T = G_3 \cos \alpha + W_1 + W_3 \quad \text{(B4)} $$

$W_1$, $W_3$ and $T$ can be eliminated from the four equations (B1), (B2), (B3) and (B4); further substitution of $\Delta L_3$ leads to $M_2$ as a function of known parameters and pile angle $\alpha$:

$$ M_2 = (G_1 L_1 + G_2 L_2 - F_p L_p) + $$$$$ + G_3 (L_3 + L_3 - h + h - l_3 \cos \alpha \cos \beta) \sin \alpha + $$$$$ - G_3 l_0 \frac{(\cos \alpha + f_3 \sin |\alpha|)}{1 - (\pm 2 f_1 T \sin \alpha |\alpha|) \sin(\alpha - \beta)} \quad \text{(B5)} $$

where: $S = \arctan \frac{q - l_3 \sin \alpha}{h - l_3 \cos \alpha}$

The $\pm$ sign in this equation to be chosen positive for an upward movement of the vessel and negative for a downward one.

In the theory above all forces resulting from current on the Puppet Weight are neglected; this simplification is accepted as negotiable in practice.
Table I

Given soil properties for North Sea Piper Field

**Soft Clay:**

- $c$ varies from 10.10 $kN/m^2$ (0.2 ksf) at mudline up to 50.10 $kN/m^2$ (1.0 ksf) at a depth of 7.5 m (25 ft).
- $y = 8.10^3 N/m^3$ (0.03 lb/in.$^3$).
- $c_0 = 2.5$

**Lateral bearing capacity:**

Resistance-deflection relationship to be obtained from API RP 2A (reference 14), paragraph 2.29 b.

- $J = 0.75$
- $D = 1.5$ m (60-in.)

This lateral bearing capacity provides the pile rotation resistance moment $M_s$ versus an increase of pile angle $\alpha$ according to the full lines in figure 6 ($M_s$ is a function of both $\alpha$ and penetration depth $z$).

**Vertical soil resistance:**

May be assumed to result from skin friction only; linear relationship for skin friction versus depth according to figure 5. Hence, $R = Ca^z$, where $C$ denotes a certain constant.

Given: pile penetrates 15 m by selfweight of pile and hammer (total weight in water 1230 KN).

(Nomenclature in this table according to API RP 2A, reference 14)

---

Table II

Data for Puppet System Operation in North Sea Piper Field

**Sea:**
- Water depth $h = 146$ m (480 ft)
- Current at surface of the water 1.4 m/s (4.6 ft/s)
- Current at sea-bottom 0.24 m/s (0.8 ft/s)
- Drag coefficient for pile and hammer 0.7

**Vessel:** Drillship Sedco 445, dynamically positioned; possible position changes $\delta q = \pm 1$ m

**Piles:**
- O.D. 1.5 m (60-in.)
- W.T. 2-in.
- Length 36.5 m (120 ft)
- Weight (above water) 84 ton (185 000 lbs)

**Hammer:**
- HBM 1220
- Weight (above water) 78 ton (172 000 lbs)
- Water displacement 28 m$^3$

**Parameters of the mechanical system (figure 3):**

- $G_1 = 500$ kN
- $G_2 = 730$ kN
- $l_s = 43.5$ m
- $l_1 = 36.3$ m
- $l_2 = 18.1$ m
- $F_d = 13.7$ kN
- $l_d = 32.4$ kN
Table III

Data for OTEC Deep Water Example

Sea: Water depth \( h = 1500 \text{ m} \) (5000 ft)
Current near the sea floor 0.3 m/s (1 ft/s)
Drag coefficient for pile and hammer 0.9

Vessel: Dynamically positioned: possible position changes 1 percent of water depth; \( \delta_{V} = \pm 15 \text{ m} \)

Piles: O.D. 1.5 m (60-in.)
W.T. 24-in.
Length 61 m (200 ft)
Weight (above water) 140 ton (310,000 lbs)

Hammer: HBM 1500
Weight (above water) 78 ton (172,000 lbs)
Weight displacement 28 m³

Puppet Weight: steel, weight (above water) 140 ton (310,000 lbs)

Parameters of the mechanical system (figure 8):

\[
\begin{align*}
G_1 &= 500 \text{ kN} \\
G_2 &= 1220 \text{ kN} \\
\theta_0 &= 1220 \text{ kN} \\
l_0 &= 67.8 \text{ m} \\
l_1 &= 62.7 \text{ m} \\
l_2 &= 30.5 \text{ m} \\
l_3 &= 5 \text{ m} \\
f_0 &= 5.34 \text{ kN} \\
f_0 &= 40.5 \text{ m} \\
f_0 &= 0.1 \\
f_0 &= 0.1
\end{align*}
\]

Table IV:

Results of analyzed OTEC Deep Water Example

<table>
<thead>
<tr>
<th>Pile landing distance ( q_1 ) 2.68 m</th>
<th>Pile landing angle ( \theta_2 ) 0.07 deg</th>
<th>upstream position change</th>
<th>downstream position change</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile standing distance ( q_s )</td>
<td>17.68 m</td>
<td>-12.12 m</td>
<td></td>
</tr>
<tr>
<td>Pile stabbing angle ( \theta_s )</td>
<td>1.2 deg</td>
<td>-1.0 deg</td>
<td></td>
</tr>
<tr>
<td>Equilibrium Position EP</td>
<td>4.2 deg</td>
<td>-5.0 deg</td>
<td></td>
</tr>
<tr>
<td>Re-Stab position upstream RS</td>
<td>9.5 deg</td>
<td>8.6 deg</td>
<td></td>
</tr>
<tr>
<td>Re-Stab position downstream RS</td>
<td>-9.5 deg</td>
<td>-9.3 deg</td>
<td></td>
</tr>
<tr>
<td>Ultimate Re-Stab position upstream URS</td>
<td>16.8 deg</td>
<td>16.1 deg</td>
<td></td>
</tr>
<tr>
<td>Ultimate Re-Stab position downstream URS</td>
<td>-16.0 deg</td>
<td>-16.0 deg</td>
<td></td>
</tr>
<tr>
<td>Smallest angle between EP and RS</td>
<td>5.3 deg</td>
<td>4.3 deg</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 1 - Schematic cross-section of the Hydroblock hammer showing the built-in buffer and the flat bottom anvil.
**METHOD 1**

Fig. 2 - The Puppet System.

**METHOD 2**

Fig. 3 - Mechanical System of Puppet System Number 1.

Stage 1: Pile and hammer just before landing on sea floor.

Stage 2: Touch-down of pile toe; a position change of the vessel is assumed.

Stage 3: Self-penetration of the pile into the sea floor.

Stage 4: Vessel changes position to obtain a vertical pile position; hammer starts operating.

**Fig. 4 - Operation of Puppet System Number 1.**

Given (table 1): skin friction linear with depth according to this figure. Hence, $R = C \cdot z$, where $C$ denotes a certain constant.

Given: if $z = 15$ m, then $R = G_1 + G_2 = 1230$ kN.

Hence, $C = \frac{1230}{15^2} = 5.47$ kN/m² (R in kN and z in m).

**Fig. 5 - Vertical Soil Resistance R versus Penetration Depth z**
Fig. 6 - Pile Rotation Resistance Moment $M_2$ versus Increase of Pile Angle $\alpha$.

- Relationship derived from soil properties
- Linearized relationship

Fig. 7 - Conservative Calculation of Pile Angle $\alpha$ versus Decreasing Hoist Force $S$ for North Sea Piper Field Project.

Fig. 8 - Mechanical System of Puppet System Number 2.

Fig. 9 - Operation of the Soil-Independent Puppet System Number 2.
FIG. 10 - PUPPET SYSTEM NUMBER 2: MOMENT M_R VERSUS PILE ANGLE \( \alpha \) FOR DEEP WATER EXAMPLE (DOCKSIDE POSITION CHANGE).

- EP = (stable) Equilibrium Position
- UEP = Unstable Equilibrium Position
- RS = Re-Stab Position
- URS = Ultimate Re-Stab Position

FIG. 11 - CAPACITIES OF PUPPET SYSTEM FOR DEEP WATER EXAMPLE.

FIG. 12 - HBM 1500 HYDROMOK HAMMER, ON BOARD OF SECCO 445 DRILLSHIP. TO ENABLE LOWERING OF BOTH AS ONE UNIT, THE PILE IS TEMPORARILY CONNECTED ONTO THE HAMMER'S PILE SLEEVE (ARROW). STANDARD 5-IN. DRILLSTRING (LEFT) IS USED AS A HOIST.

FIG. 13 - APPROXIMATELY HALFWAY THE PILE LENGTH THE HEAVY ANCHOR CHAIN (ARROW) IS PRECONNECTED TO THE 60-IN. O.D. ANCHOR PILE.
**Fig. 14** - Special fail-safe shear-off connectors (lugs on anchor pile shown by arrows) prevent hammer and pile from loosening during their mutual descent. As soon as the pile-driving starts, the connectors will be sheared off, allowing the hammer to be retrieved.

**Fig. 15** - A Puppet System operation in the North Sea Piper Field. The pile's verticality has successfully been measured using the Regan Bubble System (arrow), mounted on top of the pile-driving hammer.

**Fig. 16** - Underwater driving of conductor pipe in early production systems.
"DEEP FOUNDATIONS - SPECIFICATIONS"

For

Presentation At

University of Wisconsin - Extension

October 8, 1980

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This session is concerned with Specifications for Deep Foundations. While I do not profess to be a specifications writer, my normal duties as Chief of the Geotechnical Engineering Department of Howard Needles Tammen & Bergendoff, Kansas City, inherently results in our Department assisting in the development of specifications for our Engineering and Architectural Divisions.

I plan to touch on the general subject of Specifications and then discuss specifications for use with the more obvious types of Deep Foundations - Caissons - Drilled Shafts, and Piles. I will draw heavily on material developed by the Construction Specifications Institute, various FHWA publications, and the projects at HNTB. Finally, I will attempt to offer a few personal comments or observations concerning Specifications for Deep Foundations.

For a project to be constructed the Architect/Engineer must precisely describe his design to bidders, to the Contractor and his sub-Contractors, and to the A/E field representatives. This requires a set of coordinated construction documents which are
either graphic or written. Graphic documents, the usual engineering/architectural drawings, show the size and extent of construction and general geometric relationships between the various construction components. Written documents fall into two broad categories: (1) Contractual-Legal, and (2) Specifications.

The Specifications set requirements for strength and other physical qualities of components, standards of workmanship for manufacturers and field installation, and guarantees of components and materials. Specifications should include some, but not necessarily all, of the following information:

- Quality
- Optional Materials & Methods
- Required Guarantees
- Required Products
- Acceptable Manufacturers
- Required Physical Properties
- Required Performance
- Type and Grade of Finish
- Fabrication Method
- Installation Method.

Specifications should supplement but not repeat information shown on the drawings. Drawings and specifications should dovetail like a jigsaw puzzle with no overlap or gaps. If drawings and specifications perfectly conformed to their ideal complementary roles, there would be no need to establish procedures to resolve conflict.
4. Sinking diagrams should show sequence of operations, time and amount of concrete increments, dredging, time and amount of addition of cofferdams, caisson displacement under each load increment, amount of free board on caisson or cofferdam, water pressure at critical points for each increment of concrete or cofferdam added.

5. Pneumatic Process - Contractor provides all plant, equipment, airlocks, hospital locks, complies with requirements of "Manual of Accident Prevention in Construction" by Associated General Contractors, also requirements of National Safety Council Industrial Safety Construction Series No. 2 pamphlet, "Safety and Health in Tunnel and Caisson Work". These manuals are mentioned for reference. The trend today is to shy away from citing specific safety codes because the courts have interpreted this to mean the engineer is responsible to enforce cited codes. Instead we say: "The Contractor is solely responsible for safety".

Contractor shall retain service of at least one licensed physician who will be readily available at all times during operations under compressed air. Provide automatic recorder in compressed airline to give permanent record of air pressure, and also install in the air lines a device for recording the amounts of carbon monoxide, and the device will give audible and visible warnings (bell and light) when approaching harmful quantities.
6. **Caisson Design** - It is usually stated the design expects that sufficient weight can be developed to overcome skin friction by dredging within the working chamber and by operating independent jets or by jetting wells or both. Caissons may be constructed on earth or sandfill enclosed with sheet piling or supported by temporary dock and lowered to river bottom for sinking. If the Contractor selects the method of floating caisson into place and lowering to bottom, an additional height of steel shell will be required and Contractor will be responsible for design of the steel shell. Payment for caisson construction will be based on plan design and quantities. Contractor may substitute alternate designs for caisson cutting edge, however, no change will be made in thickness of caisson skin plate or arrangement of dredging wells.

7. **Cofferdams** - The temporary cofferdam and bracing above top of caisson shall be designed by Contractor and the design examined by the Engineer.

8. To ensure location of pier caissons within the limits specified, care must be exercised in sinking caissons. Cutting edges must be set on exactly correct horizontal position and be maintained at true level. Sink caissons so that the center of the caisson at the cutting edge will be within a circle 15 inches in diameter,

9. Contractor shall make all excavations of every nature in whatever materials encountered. No blasting shall be done without written approval of the Engineer.

10. Dispose of material excavated from caissons in accordance with agency requirements.

11. A competent diver shall be available at all times to assist in removing obstructions and to check for scour during construction and to check final cleaning before sealing.

12. Plans show assumed founding elevations, however, Contractor should be prepared to continue sinking until satisfactory founding conditions are reached.

13. In preparing for the tremie seal, all material below the high interior cutting edges must be removed. All clay, silt, boulders, or foreign materials shall be removed from walls and interior of dredging chambers prior to placing tremie seal.

14. Following sealing of caissons, tight covers are placed over all dredge wells, and when directed by the Engineer, the Contractor will fill the pier with water to the approximate low water elevation.
B. **DRILLED SHAFTS** — Sometimes referred to as **Drilled Piers, Drilled Caissons, or Large Diameter Bored Piles, Cast-in-Situ Bored Piles.**

A DRILLED SHAFT IS A DEEP FOUNDATION CONSTRUCTED BY PLACING CONCRETE, USUALLY WITH REINFORCING STEEL, IN A MACHINE EXCAVATED CYLINDRICAL HOLE, FOR THE PURPOSE OF TRANSFERRING STRUCTURAL LOADS TO BEARING STRATA WELL BELOW THE USEABLE PORTION OF THE STRUCTURE.

They are constructed by the Dry Method, Casing Method, and Slurry Displacement Method and good specifications covering the method of installation are necessary to produce an acceptable drilled shaft installation. The specifications for Drilled Shafts should deal with critical aspects of the construction process and would include:

1. **Inspection provisions for adequate inspection of the excavation prior to placing steel and concrete.**

2. **Reinforcing Steel** — If used the rebar cage consisting of longitudinal bars and spiral reinforcement, lateral ties, or horizontal bands should be completely assembled prior to placement. Standard specifications for rebar steel will be cited. For full length reinforcement, at least one-half of the longitudinal bars required in upper portion of the shaft will be extended to bottom with proper lateral reinforcement. Tack-welding may be used.
IN ATTACHING SPIRAL REINFORCEMENT, BANDS, OR LATERAL REINFORCEMENT; HOWEVER, WELDING WILL NOT BE PERMITTED OVER TOP PORTION OF REBAR CAGE. THIS DEPTH CAN BE DETERMINED BY ANALYSIS OF SHAFT UNDER LATERAL LOADING.

3. Concrete - Design and Mixing - Items to be covered include:
   - Required physical properties of aggregate, cement, mixing water.
   - Allowable temperature of these materials.
   - Additions such as retarders or air-entraining agents.
   - 28-Day Compressive Strength.
   - Maximum size of coarse aggregate.
   - Clear spacing between rebar, or between rebar cage and inside of hole or casing should be at least three times maximum size of coarse aggregate.

4. Concrete Workability -
   This is usually specified by the slump of concrete which at time of placement should be no less than 4". It will more than likely be around 6" if contractor can demonstrate good quality concrete is being obtained.

5. Concrete - Time between Mixing and Placing -
   Some specifications give a limited time period, usually 1 Hour, between the mixing of concrete and its placement. For some jobs this may be too short a time. Mixes can be designed so that several hours can elapse between mixing and final placement. The specification for design of
CONCRETE, WITH REGARD TO TIME OF PLACEMENT SHOULD BE OF
THE PERFORMANCE TYPE, AND MERELY REQUIRE THAT THE QUALITY
OF CONCRETE IN THE HOLE MEET APPROPRIATE STANDARDS.

6. **Slurry** — CURRENTLY THERE IS NO DATA AVAILABLE THAT WILL
ALLOW THE UNIT WEIGHT OF THE DRILLING FLUID TO BE SPECIFIED.
A PERFORMANCE SPECIFICATION IS THEREFORE DESIRED AND
MIGHT READ AS FOLLOWS:

"The density of the drilling fluid employed in
advancing an excavation in a caving formation shall be
such that the formation is stabilized. Furthermore, the
density of the slurry shall be such that it is fully
displaced by fluid concrete. The contractor may employ
bentonite or some other additive, and it may be necessary
for circulation to be employed to maintain the quality of
the drilling fluid (and suspension) during construction,
and/or just prior to the concrete pour."

7. **Placing Reinforcing Steel** — THIS SPECIFICATION SHOULD
MERELY REQUIRE THAT THE REBAR CAGE OR DOWELS BE PLACED
IN THE DRILLED SHAFT IN AN UNDAMAGED CONDITION AND
ACCORDING TO PLAN.

8. **Placing Concrete** — SPECIFICATIONS FOR THIS PHASE OF
DRILLED SHAFT CONSTRUCTION ARE CRITICAL. IMPROPERLY
WRITTEN SPECIFICATIONS CAN LEAD TO A CONSIDERABLE INCREASE
IN COSTS, OR TO POOR QUALITY OF CONSTRUCTION.
A) **Placing Concrete by Free-Fall Method** -

Concrete can be placed by Free-Fall if it falls into its final position through air without striking the sides of the hole, the rebar cage, or any other obstruction.

B) **Vibration or Rodding** -

Complete vibration or rodding of concrete is not required. Some minor rodding near the outside of the shaft is necessary to eliminate honey-combing where the hydrostatic pressure in the fresh concrete is low. Rodding is necessary only in the top 3 Ft. of the shaft.

C) **Placing Concrete in Casing & Pulling Casing** -

If the casing method of construction is employed, the concrete must be brought above the level of the external fluid before the casing is pulled. The top of the casing shall be at ground surface or above. The hydrostatic pressure in the concrete column shall be greater at all times than the pressure in any column of fluid trapped behind the casing. Thus, the drilling slurry will be expelled from the excavation as the casing is pulled. Because the concrete column will slump as the casing is pulled, it will be necessary in some cases to add fresh concrete in the top of the partially-pulled casing to ensure that the concrete column is at the proper height when the casing is completely extracted.
D) **Slump of Rebar Cage When Casing is Pulled**

Many specifications require the contractor to hold the top of the rebar cage during the pulling of the casing and that the top of the cage not rack or move downward more than a small amount. Such a procedure is possible, but not very practical. An extra crane and crew would be needed, and perhaps they would work only during the pulling of casing. The crane holding the rebar cage would have to be very large, because the casing would have to be lifted along the line to the top of the cage. A more practical procedure is to use the slurry displacement method of construction where the concrete column is moving up with respect to the rebar cage, or to design the cage as a structure that will resist downward forces from the column of fluid concrete. Design of the rebar cage of proper structural characteristics is difficult because the shearing strength of the column of fresh concrete is not well known, nor are the torsional and buckling characteristics of the rebar cage. A solution normally used consists of welding horizontal bands (2" in width and about 3/8" in thickness) to the rebar cage at intervals of 5 feet, and over the lower portion of cage below the zone of significant bending moment. The workability of the concrete must be good and the casing must not be pulled too fast, in order to minimize the downward force.
E) \textbf{Placing Concrete Under Slurry or Under Water -}

If the slurry displacement method is employed, or if the Engineer has allowed the Contractor to place the concrete under a column of water, care must be taken to ensure that all the fluid is expelled from the hole. The concrete must be placed by a tremie or pumped. In order to prevent contamination of the concrete placed initially, the bottom of the tremie pipe is sealed with a diaphragm or plug that is flushed out when the hydrostatic pressure from the column of concrete exceeds that of the fluid in the hole. An acceptable alternative procedure is to employ a plug that will move down the pipe to keep the concrete separate from the slurry. An inflatable ball has been used for this purpose. The tremie must be water-tight, the concrete must have good flow characteristics, and the bottom of the tremie, or pump pipe, must always remain below the top of the column of fluid concrete. The concreting should always be carried out in a continuous operation.

F) \textbf{Inspection of Concrete -}

There is one aspect of the inspection of fresh concrete that requires mention. In some cases a considerable amount of time can be expended while inspectors take test cylinders, perform slump tests, and measure air entrainment. The trucks are standing by, the Contractor is waiting, and the concrete is hydrating.
The Engineer should do as much inspection as possible at the plant site. If he requires a period of 15 to 20 minutes to perform tests of each truck of concrete delivered to the job, the specifications should so state.

C. Piling

State Departments of Transportation are possibly the largest agency users of foundation piling and over many years have developed practical specifications for piling, applicable to their geographical location. Piling installation contractors are, or should be, familiar with these standard specifications.

In our practice the specifications for foundation piling are frequently furnished by the Owner and we need only to adapt the specifications to the intended construction, usually with Special Provisions or Supplemental Specifications.

For a private, non-agency client, we customarily pattern specifications for their piling jobs after those in current local usage, usually the State DOT Specifications. Some Geotechnical Engineering Firms, and Architects/Engineers prefer to write "their own" piling specifications, however, if you will check them closely, you will find great similarity to local "standard" DOT Specifications.

For this discussion I have selected as reference specifications those in current use by the States of Louisiana and Texas, where several types of end bearing and/or friction piles are installed.
IMPORTANT ITEMS TO COVER IN PILING SPECIFICATIONS INCLUDE:

1. **Materials:**
   
   a. **Precast Concrete Piles** -
      
      Re-Steel, usually ASTM A 615,
      
      Concrete - 3000 psi compressive strength.

   b. **Prestressed Piling** -
      
      Concrete - generally 5000-psi compressive strength,
      Prestressing Steel - ASTM A 416, or A-421, Uncoated stress-relieved strands.
      The design, mixing, placing, curing, quality of concrete construction, removal of forms, prestressing details, tensioning and release of stress and inspection facilities are very important to production of an acceptable product.

   c. **Cast-In-Place Concrete Piles** - (sometimes referred to as Metal Shell Piling).
      
      Metal Shell Piling - ASTM A-252, Grade 2, or A-36.
      Concrete - usually 3000 psi compressive strength
      Re-Steel - (where required) usually ASTM A-615.

   d. **Steel Piling** -
      
      Steel (H Section & Sheet) - ASTM A-36.

   e. **Timber** - ASTM D-25
      
      Untreated Piling - any species of durable timber that will satisfactorily stand driving.
      Treated Piling - Southern Pine or Douglas Fir impregnated with a preservative.
Specifications are usually lengthy, including discussions on: Knots, Checks, Splits, Shakes, Density, Peeling & Trimming, Soundness, Seasoning, Straightness.

2. Preparation for Driving

A. Excavation - Piles are not usually driven until excavation is complete.

B. Embankment - Construct embankment at bridge ends to elevation of bottom of abutment bent cap prior to driving piles in that unit.

C. Transportation-Precast Concrete Piling - Precast - prestressed piles are to be supported at each of the pickup points as shown on plans for particular length of piles. Supports not more than 1 foot from theoretical position for each support, and distance between 2 supports will not be more than 1 foot from theoretical.

D. Support Holes - When approved by Engineer

Piles may be set in supporting holes - 10 feet or less for piles up to 50 feet in length, or 20% of designated penetration for piles over 50' in length. Fill hole around pile with granular material. This is usually a cosmetic item for after a few shovels of sand the hole, if open to start with, is sealed off and rodding is not usually performed.
3. **Driving Piling**

   A. **Tolerances** - Bents - Transverse to C, within 2" of plan.  
      Parallel to C bent - within 4" of plan.  
      **Foundation Piling** - Top not more than 4"  
      in any direction from plan position.  
      **Cutoff** - 2" of plan cut-off grade.

   B. **Sizes of Driving Equipment.**  
      There are some general rules for selecting Driving Equipment, based on Pile Types as follows:

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Min. Hammer Energy, Ft.-Lbs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber</td>
<td>330 R</td>
</tr>
<tr>
<td>Steel H</td>
<td>250 R or 2½ Wp (larger of two)</td>
</tr>
<tr>
<td>Metal Shell</td>
<td>350 R (for length &gt; 65', 430 R)</td>
</tr>
<tr>
<td>Metal Shell w/Mandrel</td>
<td>220 R but not less than 1-Ft-Lb, per lb. pile weight, including mandrel.</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td>250 R but no less than 1-Ft.-Lb per lb. pile weight.</td>
</tr>
</tbody>
</table>

   R = Design Load in Tons  
   Wp = Weight of Plan Length Pile, in pounds.

   C. **Pointing** - Timber or steel bearing pile to be pointed as soil conditions require.
D. **Splicing**

Precast Piles - Furnished and driven in full lengths.

Cast-In-Plast Concrete Pile Shells - can be field spliced but not in too short sections. Use manufacturer's recommendations to satisfaction of Engineer.

Steel Bearing Piles - furnished and driven in full lengths. If splice authorized, not more than 2 per pile by welding.

Timber - Furnish and drive full length where practicable. Splicing only by written permission, in accordance with splicing detail furnished or approved by Engineer.

E. **Painting** - Normally no painting required except where steel or steel shell piles extend above ground.

F. **Pilot Holes**

Holes with diameter equal to 7/8 the face width of a square pile, or 7/8 average diameter of a round pile, but shall be of a size that will provide desired results. Fill open space with granular material. In no case shall pilot hole extend within 5' of tip elevation of pile.

G. **Leads - Templates** - Fixed or swinging leads may be used.

Swinging leads used in combination with rigid template. Inclined leads used in driving batter piles.

H. **Followers & Underwater Hammers** - Permitted only by written approval of Engineer. When follower or underwater hammer is used, 1 pile in each group of 10 shall be furnished long enough to permit being driven without a follower or under-
WATER HAMMER AND SHALL BE USED BY A TEST PILE TO DETER-
MINE AVERAGE BEARING OF GROUP.

1. **Water Jets** - Size of jets, volume and pressure of water at jet
nozzles sufficient to erode material adjacent to pile. 150 PSI pressure at two 3/4" jet nozzles is probably a
minimum. Withdraw jets as drive with hammer to secure
final penetration. No jetting within 5 feet of tip
elevation of piles unless authorized. Water jets will not
be permitted where embankment stabilization or other
improvements would be endangered.

J. **Interrupted Driving** - For interruption before reaching final
penetration, the record for resistance shall not be taken
until at least 12 inches of penetration has been obtained
after driving resumed.

K. **Protection of Pile Heads** - Nature of driving may require
protection for heads of concrete and timber piles. Use
approved cap, having rope or other cushion next to pile
head and fitted into a casting which supports a timber
shock block.

L. **Cut-Offs Precast Concrete Pile** - Make cutoff perpendicular
to pile axis at elevation shown. Avoid spalling of
concrete. Re-steel remains to engage body of footing
or cap.

- **Steel Bearing Piles** - Cut at right angles to axis of pile.
- **Timber Piles** - Saw at right angles to axis.

Pile supporting timber caps should be sawed to
horizontal plane.

Shimming on tops of piles not permitted.
- Cast-in-Place Concrete Piles - After fully driven, inspected, and approved, but neatly at right angles to pile axis.

M. Extension of Precast Concrete Piling - By casting in place or by splicing a precast section.

4. Bearing Resistance - Usually determined by:

A. Hammer Formula Method - Most DOT specifications utilize the ENR formula:

\[ P = \frac{2WH}{S+1.0} \] (Gravity Hammers)

\[ P = \frac{2WH}{S+0.1} \] (Single Acting Hammer)

\[ P = \frac{2E}{S+0.1} \] (Double Acting Hammer)

B. Wave Equation Method - If specified on plans that bearing capacity will be determined by Wave Equation Method contractor shall submit to engineer, following:

- Manufacturer's specification data for proposed hammer.
- Complete description and dimensions including total thickness of all cushioning material used between pile and helmet.
- Complete description and dimensions including total thickness of cushioning material in cap block including direction of grain if wood is used.
5. **Bearing and Length Determination**

A. **Penetration Data** - Piles should be driven to "minimum", or "required" penetration as given in plans and specifications.

B. **Test Piling** - Test Piling is driven and with the pertinent hammer formula or wave equation bearing graph to determine bearing resistance. Approved lengths are determined.

C. **Test Load** - When required by plans, piling shall be driven and test loaded. Appropriate hammer formula used to establish dynamic resistance of test load pile and anchor piles. Test Load data is used to determine pile lengths and to develop "K" factor to modify hammer formula.

\[
K = \frac{L}{P}
\]  

Where

- **K** = Static correction factor to a dynamic formula.
- **L** = Maximum safe static load proven by test load.
- **P** = Dynamic resistance of test loaded pile determined by hammer formula.

D. **Soil Tests** - Piling are driven to tip elevations shown on plans. Dynamic resistance determined by hammer formula. Subsequent piling driven to approved tip elevations to obtain required bearing determined by hammer formula modified by K factor or to maximum penetration shown on plans.
K = \frac{R}{P} \text{ when}

K = \text{ Static correction factor to a dynamic formula}
R = \text{ Maximum safe static load predetermined by soil studies.}
P = \text{ Dynamic resistance of piling by hammer formula.}

6. **Test Loads** - Over the last several years, we have routinely recommended to clients that the so-called "quick" load test be performed. This procedure developed out of the Constant Rate of Penetration (CRP) test in 1961. This method requires load increments be applied in 5 to 10-Ton increments with load, gross settlement, and other pertinent data recorded immediately before and after the addition of a load increment. The load is maintained constant for 2½ minutes before the next increment is added.

This permits a load test to be performed in a matter of only several hours as compared to other methods, ASTM D 1143 which might take well in excess of 48 hours.
D. COMMENTS -

0 Realistic, enforceable specifications are necessary to achieve quality construction. Adequate time should be made available for the drafting and editing of specifications, tailored for specific objectives. You will have better specifications if you complete the plans or working drawings in time for the specification writers to study them and discuss the intended construction with those involved in design and plans.

0 There is a tendency to utilize Master File Specifications. This can be a time-saver if the so-called Master Specifications are properly adapted for specific objectives. All too often we see job specifications straight from the files with little or no attempt to relate them to the site specific conditions. This will get you in trouble.

0 In our design practice, we routinely discuss difficult construction problems or concerns with local, competent, construction firms, during the design and plan stage. The Contractor's influence at that stage of the project can more than offset any embarrassment to you by asking questions.
The current trend is toward utilization of the Wave Equation Analyses in design and construction of pile foundations, and there are many projects where this is beneficial to the Owner. I believe this procedure is being over sold. There are many pile foundations where the Wave Equation is not needed in design and where adequate documentation of pile installations utilizing the conventional hammer formulas will be quite adequate.

High capacity pile load tests involving 1000 Tons are quite common now, particularly in the axial test loading of single drilled shafts.

Handling loads of this magnitude by hydraulic jacks can be very dangerous and the specifications should provide for added precautions relating to equipment and systems, and working in the area.

Loading should be performed in normal daylight hours, if at all possible. If not, the specifications must clearly address the matter of adequate lighting being ready at the site before any loading is permitted.

In high capacity load testing of pile and drilled shafts, the specifications should require all load-deflection readings be by transit located at a safe distance. Direct reading of the dials beneath the loading frame can be a dangerous assignment.
The Quick Load Test procedure for piles and drilled shafts is an acceptable procedure and reduces the cost of test loading considerably as the time required may be 1 to 3 hours in lieu of the 48-hour procedure.

Problems frequently develop where a large number of displacement piles are driven, at minimum spacing, in a cofferdam. Cofferdam movements, always a concern, can be very disturbing. The specifications should require the Contractor to submit to the Engineer an acceptable driving sequence; along with his shop drawings and design for the cofferdam.

In drilled shaft work where there is a dry hole and good stability, we find that Contractors prefer to delay-concreting until late in the day. This is particularly true where the drilled shaft Sub-contractor only excavates the hole, and the Prime or General Contractor places the steel and concrete. Specifications should be written to encourage the Contractor to follow-up the shaft drilling closely with the concrete. Night pours should be avoided.